RAINFALL-INDUCED SLOPE FAILURES AND PREVENTIVE MEASURES IN SINGAPORE

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Geotechnical Engineering Monograph
RAINFALL-INDUCED SLOPE FAILURES AND PREVENTIVE MEASURES IN SINGAPORE

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Nanyang Technological University
Housing & Development Board

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The first figure (top left) on the cover shows the mechanisms of rainfall-induced slope failure. The second figure (top right) on the cover presents an example of rainfall-induced slope failure that occurred in Singapore during heavy rainfall in December 2006. The third figure (bottom left) on the cover shows the schematic diagram of capillary barrier system as slope preventive measure against rainfall-induced slope failure. The fourth figure (bottom right) on the cover shows the layout of field instrumentation of capillary barrier system and vegetative cover in Singapore.

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All informations (drawings, tables, charts, models, designs, specifications, photographs, computer software, surveys, calculations and other data) provided in this monograph are compiled based on previous laboratory tests, field tests, field instrumentations and numerical analyses of the corresponding soil types for specific climatic conditions, geological features, topography and vegetation of the investigated slopes. The authors do not guarantee, warrant, or make any representation regarding the use of, or the results of, the analyses and design in terms of correctness, accuracy, reliability, currentness, or otherwise; the user is expected to make the final evaluation in the context of his (her) own engineering problems.

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The project was carried out by the School of Civil and Environmental Engineering (CEE), Nanyang Technological University (NTU) and Building & Infrastructure Group (BIG), Housing and Development Board (HDB). The excellent cooperation and coordination between NTU and HDB has resulted in the successful completion of the project. All the objectives and deliverables have been achieved within the timeframe and budget of the project. NTU has been studying rainfall-induced slope failures for a number of years and has made significant strides in understanding the mechanisms associated with rainfall-induced slope failures. BIG, with its portfolio of overseeing public housing construction has extensive experience in undertaking remedial works on rainfall-induced slope failures. The collaborative research was carried out in such a way as to leverage the combined strengths of these two organizations.

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EXECUTIVE SUMMARY

Steep residual soil slopes are generally characterized by a deep ground water table and a significant thickness of unsaturated soil above the water table. The negative pore-water pressure (matric suction) above the water table provides additional shear strength to the unsaturated soil. The unsaturated soil zone is also influenced by the ground surface moisture flux boundary condition changes (i.e., infiltration, evaporation and transpiration). As rain water infiltrates into slope, the matric suction in the soil decreases and as a result, the apparent shear strength associated with matric suction can decrease, causing the slope to become susceptible to failure. The factors which influence slope instability include the geology, topography of the area, soil types, soil properties, local climates (rainfall and evaporation) and water flow patterns within the slope. Generally, the effect of rainfall in inducing slope failures is attributed to the loss of matric suction as the wetting front advances from the ground surface or to the rise in the ground water table within the slope. It is also possible that both processes that occur simultaneously.

Rainfall-induced slope failures are common in tropical areas such as Singapore that are covered by residual soils. These failures can pose potential danger to infrastructures and public safety. Therefore, a comprehensive study of the mechanisms associated with rainfall-induced slope failures, the stabilization, protection and repair works of failed slopes is needed. The Housing and Development Board (HDB) has partnered Nanyang Technological University (NTU) to undertake research studies with the objective of better understanding, and providing engineered solutions to rainfall-induced slope failure problems in HDB estates and to investigate the effectiveness of possible novel preventive measures. Unsaturated soil mechanics principles were incorporated into the research programs. The research methodology involved, firstly, the determination of relevant unsaturated soil properties together with field instrumentation and the monitoring of several slopes in Singapore. At the same time, computer modeling was undertaken to simulate seepage through unsaturated and saturated soil system in order to incorporate the effects of rainfall on slope stability.

The objectives of the project was to study the effects of soil properties and climatic conditions such as rainfall on potential slope failures in Singapore. Thirty-one slopes were selected which consisted of eleven sites located in residual soils from sedimentary Jurong Formation, ten sites from Bukit Timah Granite and another ten sites from Old Alluvium. Site investigations including soil sampling and field tests were conducted to obtain undisturbed samples. Infiltration tests and
In-situ permeability tests were performed to obtain the permeability characteristics of the soils from the selected sites. The basic soil properties (i.e. classification and volume-mass properties), saturated and unsaturated hydraulic properties, and saturated and unsaturated shear strengths were obtained from laboratory tests.

Some slopes were instrumented to measure the pore-water pressure response during dry and wet periods. Infiltration seepage modelling was performed using a saturated-unsaturated seepage finite element program to understand the pore-water pressure distributions in residual soil slopes under various surface flux boundary conditions. The model results were further verified using data collected from the instrumented slopes. In addition, the stability of residual soil slopes was assessed using a slope stability computer program that took into consideration the geometry, stratigraphy, unsaturated and saturated soil properties in addition to the pore-water pressure distributions computed from the seepage model simulations.

With the information obtained from slope stability analyses, risk and hazard assessment analyses were carried out for all the investigated slopes. Recommendations for slope improvement works were provided for various levels of risk and hazard.

Preventive measures against potential slope instability were investigated at several residual soil slopes. These methods included the installation of additional drainage systems, construction of capillary barrier systems and the planting of deep-rooted vegetation.

The effectiveness of a horizontal drainage system to maintain negative pore-water pressure is governed by soil properties, slope geometry, drain type, location, number, length and spacing. Rainwater infiltrating into the slope contributes to raising the groundwater table and increasing pore-water pressures. Therefore, it is of importance in some situations to install horizontal drains near the toe of the slope to lower the groundwater table and consequently lower the pore-water pressures.

The proposed capillary barrier systems were designed based on the differences in unsaturated hydraulic properties between two layers in the system. Under unsaturated conditions, the difference in permeability between the fine-grained layer and the coarse-grained layer limits the downward movement of water. This behavior is called the capillary barrier effect where water is stored and transported within the fine-grained layer. The infiltrating water is ultimately removed by lateral drainage through the slope, evaporation and transpiration.
Green technology combines vegetation and engineering design methods to mechanically reinforce slopes, control erosion, improve aesthetics of the environment, provide visual and noise barrier and improve biodiversity. When the roots of the vegetation interact with the soil, a new composite material with root-reinforcement is formed. The end result is an increase in the shear strength of the soil and a change in other mechanical properties.

The effectiveness and suitability of various preventive measures were evaluated with respect to different soil types in Singapore.
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Introduction

Background

Rainfall-induced slope failure is a common problem in many tropical areas that are covered by residual soils. Furthermore, rapid growth of regional economies has resulted in tremendous demand for hillside developments involving engineered and fill slopes. Rainfall-induced slope failures can pose potential danger to infrastructures and public safety. In addition, the repairing of failed slopes can be quite costly.

Factors influencing slope instability due to rainfall include the geology and topography of the area, soil types, soil properties, local climatic conditions (rainfall and evaporation) and water flow patterns within the slopes. The incidences of slope failures in Singapore during periods of intense rainfall demonstrate the crucial influence of rainfall and the subsequent movement of groundwater in slopes. Therefore, the Housing and Development Board (HDB) has partnered with Nanyang Technological University (NTU) in solving rainfall-induced slope failure problems around HDB estates. The mechanisms associated with such slope failures needs to be properly understood in order that preventive measures can be utilized. Preventive measures may involve the installation of drainage systems, use of “green” engineering soil cover, planting of deep-rooted vegetations, and the construction of capillary barriers. All of the potential measures need to be investigated under Singapore’s equatorial climatic conditions.

Objectives

1) To study the relationships between soil types and soil properties with respect to the potential for slope failures in Singapore.

2) To establish the correlation between local climatic conditions (rainfall, infiltration and evaporation) and slope failures in Singapore. Slope instability needs to be understood in terms of the response of the soil to pore-water pressure changes and slope deformations.

3) To study the variations of soil properties and typical changes in pore-water pressures with respect to climatic conditions (rainfall and infiltration) in Singapore.

4) To evaluate the stability of representative slopes in Singapore under typical rainfall conditions in Singapore
5) To establish guidelines for several new preventive measures for slope failures with respect to cost efficiency, ease of installation and suitability for use with different soil types in Singapore.

Methodology

The research involved five main parts; namely, site investigation, laboratory works, field instrumentation, modelling and assessment of preventive measures.

1) Site investigation included soil sampling and field tests. Several soil slopes in Singapore were selected for detailed study. The sites were investigated by drilling at least three bore holes at each site and obtaining undisturbed samples. The infiltration test and in-situ permeability tests were performed to obtain the permeability characteristics of soil.

2) A comprehensive laboratory programme was conducted to characterize soil basic properties, hydraulic properties, and shear strength. Saturated and unsaturated soil tests were conducted in the laboratory.

3) Selected slopes were instrumented with pore-water pressure measuring devices (i.e., tensiometers and piezometers) and rainfall gauges were installed in order to study the pore-water pressure response during dry and wet periods. The effects of rain water infiltration on slope movements was investigated by installing deformation measuring devices. Data acquisition systems were used for automatic recording and supplemented by manual checks at occasional intervals.

4) Flow modelling was performed using a saturated-unsaturated seepage, finite element computer program. The geological, hydrogeological, climatic, topographic and geotechnical properties of the problem were integrated in the model. The model provided insight to the pore-water pressure distributions in residual soil slopes under various ground surface moisture flux boundary conditions. The flow modelling was verified using the data collected from the instrumented slopes.

5) The assessment of stability of residual soil slopes was performed by incorporating the results from the flow modelling into a slope stability analysis. The combined seepage and slope stability analyses provided information on the fluctuation of factor of safety for slopes subjected to various rainfall and soil cover conditions.
6) Appropriate slope stabilization methods were recommended based on the new approach of slope stability analyses.

Site Investigation and Field Tests

Slope Selection

Thirty-one slopes in Singapore were selected for site investigations which are located in the major formations in Singapore, i.e., Jurong Formation (JF), Bukit Timah Granite (BTG) and Old Alluvium (OA). The locations of the investigated slopes are shown in Figure 1. The investigation of slopes was divided into three batches. Batch 1, batch 2 and batch 3 comprised site investigations of 11, 10 and 10 slopes, respectively.

Figure 1: Location of Investigated Slopes in Singapore (after Rahardjo, 2014a)

Topographical Surveys

Topographical surveys were performed to establish the geometry of the selected slopes. The simplified geometry of each slope from batches 1, 2 and 3 are summarised in Tables 1 to 3. The height of the investigated slopes in batch 1 varied between 5 to 15 meters. The angle of the investigated slopes in batch 1 varied from 27° to 39°.

The angle of the investigated slopes in batch 2 varied from 26° to 32° which were gentler than those in batch 1. The heights of the investigated slopes in batch 2 were similar to those in batch 3 which were between 7 m to 20 m. The geometries of the slopes in batch 3 were similar to the slopes in batch 1. The angles and the heights of the slopes in batch 3 were between 27° to 33° and between 5 m to 20 m, respectively. The relationship between slope height and
slope angle for residual soil slopes from JF, BTG and OA investigated in this project are presented in Figure 2.

Table 1: Simplified geometry of slopes at batch 1 (after Rahardjo, 2014a)

<table>
<thead>
<tr>
<th>No.</th>
<th>Name of Slope</th>
<th>Formation</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Length (m)</th>
<th>Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slope 1-1</td>
<td>Jurong Formation</td>
<td>9.1</td>
<td>21.0</td>
<td>22.9</td>
<td>29.4</td>
</tr>
<tr>
<td>2</td>
<td>Slope 1-2</td>
<td>Jurong Formation</td>
<td>8.2</td>
<td>18.0</td>
<td>19.8</td>
<td>27.4</td>
</tr>
<tr>
<td>3</td>
<td>Slope 1-3</td>
<td>Jurong Formation</td>
<td>11.7</td>
<td>19.0</td>
<td>22.3</td>
<td>33.0</td>
</tr>
<tr>
<td>4</td>
<td>Slope 1-4</td>
<td>Bukit Timah Granite</td>
<td>11.6</td>
<td>21.0</td>
<td>24.0</td>
<td>28.9</td>
</tr>
<tr>
<td>5</td>
<td>Slope 1-5</td>
<td>Bukit Timah Granite</td>
<td>8.9</td>
<td>20.0</td>
<td>21.9</td>
<td>27.1</td>
</tr>
<tr>
<td>6</td>
<td>Slope 1-6</td>
<td>Old Alluvium</td>
<td>9.2</td>
<td>20.0</td>
<td>22.0</td>
<td>27.7</td>
</tr>
<tr>
<td>7</td>
<td>Slope 1-7</td>
<td>Bukit Timah Granite</td>
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<td>29.0</td>
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<td>29.2</td>
</tr>
<tr>
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</tr>
<tr>
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<td>28.4</td>
<td>31.5</td>
<td>27.8</td>
</tr>
<tr>
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<td>7.0</td>
<td>31.5</td>
<td>39.1</td>
</tr>
</tbody>
</table>

Table 2: Simplified geometry of slopes at batch 2 (after Rahardjo, 2014a)

<table>
<thead>
<tr>
<th>No.</th>
<th>Name of Slope</th>
<th>Formation</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Length (m)</th>
<th>Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slope 2-1</td>
<td>Bukit Timah Granite</td>
<td>11.7</td>
<td>34.0</td>
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<tr>
<td>2</td>
<td>Slope 2-2</td>
<td>Bukit Timah Granite</td>
<td>18.4</td>
<td>56.0</td>
<td>59.0</td>
<td>28.6</td>
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<tr>
<td>3</td>
<td>Slope 2-3</td>
<td>Bukit Timah Granite</td>
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<td>43.0</td>
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<td>Bukit Timah Granite</td>
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<td>28.0</td>
<td>29.0</td>
<td>26.4</td>
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<tr>
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<td>Bukit Timah Granite</td>
<td>11.6</td>
<td>41.0</td>
<td>43.2</td>
<td>31.7</td>
</tr>
<tr>
<td>6</td>
<td>Slope 2-6</td>
<td>Jurong Formation</td>
<td>11.9</td>
<td>24.0</td>
<td>26.9</td>
<td>27.7</td>
</tr>
<tr>
<td>7</td>
<td>Slope 2-7</td>
<td>Old Alluvium</td>
<td>6.9</td>
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<td>18.3</td>
<td>27.0</td>
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<tr>
<td>10</td>
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<td>Jurong Formation</td>
<td>47.1</td>
<td>97.0</td>
<td>108.2</td>
<td>29.6</td>
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</table>
Table 3: Simplified geometry of slopes at batch 3 (after Rahardjo, 2014a)

<table>
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<tr>
<th>No.</th>
<th>Name of Slope</th>
<th>Formation</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Length (m)</th>
<th>Angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Slope 3-1</td>
<td>Bukit Timah Granite</td>
<td>7.3</td>
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<td>27.6</td>
</tr>
<tr>
<td>2</td>
<td>Slope 3-2</td>
<td>Old Alluvium</td>
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<td>6.3</td>
<td>8.7</td>
<td>33.0</td>
</tr>
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<td>3</td>
<td>Slope 3-3</td>
<td>Jurong Formation</td>
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<td>12.6</td>
<td>15.5</td>
<td>32.9</td>
</tr>
<tr>
<td>4</td>
<td>Slope 3-4</td>
<td>Jurong Formation</td>
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<td>19.7</td>
<td>22.0</td>
<td>32.0</td>
</tr>
<tr>
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<td>Slope 3-5</td>
<td>Jurong Formation</td>
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<td>28.5</td>
<td>30.4</td>
<td>27.6</td>
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<td>Slope 3-6</td>
<td>Jurong Formation</td>
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<td>Slope 3-7</td>
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<td>20.3</td>
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</tr>
</tbody>
</table>

Figure 2: Variation of slope heights and angles for the investigated slopes in Singapore (after Rahardjo, 2014a)

Sampling

Sampling was conducted by borehole drilling to obtain undisturbed samples at each site. In total, 38, 46 and 43 boreholes were drilled for the slopes in batch 1, batch 2 and batch 3, respectively. All boreholes were drilled using a rotary drilling rig. Drilling was terminated if rock was encountered or the SPT N value was already higher than or equal to 50 or the drilling rod already reached a depth of 5 meters below the toe of the slope. The average depths for batch 1, batch 2, and batch 3 were 14, 10.7 and 12.6 meters, respectively.

The boreholes were advanced using rotary wash boring and triple core barrel method to obtain more representative undisturbed samples. If hard soils were encountered, samples were collected using hammer driving of an open drive.
sampler. Diamond coring was performed in the boreholes when rock material was encountered. The diamond coring was used to verify the existence of rock boulders, bedrock and the quality of the hard material. Representative samples were also taken from the split spoon sampler for soil description and classification.

Foam liquid was used for drilling the borehole in order to minimize friction between the sample and the tube, to maintain the water content of the undisturbed sample and to improve the stability of the hole. The undisturbed samples from these boreholes were taken continuously using a Mazier sampler until the required depth of boreholes was reached.

Rock coring was also carried out in some boreholes. Rock Quality Designation (RQD), colour, fracture condition, and weathering grade were recorded before the samples were sent to the laboratory.

**Standard Penetration Tests**

Standard penetration tests (SPT) were carried out at 1 m interval in the boreholes. A total of 29, 21 and 30 SPT tests were conducted for batches 1, 2 and 3, respectively.

**Permeability Tests**

Rising-head permeability tests were conducted in the piezometer holes. The test was performed by lowering the water level inside the piezometer pipe. The water level was reduced sufficiently by bailing out the water from the pipe. The water level depth was recorded at regular time intervals up to 5 to 7 hours. The schematic diagram of the rising head permeability test is shown in Figure 3.

**Infiltration Tests**

Infiltration tests were performed using a double-ring infiltrometer. This test was conducted to obtain the infiltration rate of the surface soil. Two tests each were carried out at Slope 1-3 and Slope 1-7.

The infiltration rate is approximately equal to the saturated permeability, $k_s$, based on the following assumptions:

1. The difference in elevation between the water level in the infiltrometer and the edge of the wetting front was assumed to be significantly greater than the drop in the water level of the infiltrometer for a given period of time.
2. The water pressure at the edge of the wetting front was zero.
The first assumption showed that the infiltration test was carried out at a constant head and steady state condition could be achieved if the change in water level of the infiltrometer was constant with time. The second assumption showed that the hydraulic gradient for the infiltration test was equal to unity and the infiltration rate was equal to $k_s$.

Figure 3: Schematic diagram of rising-head permeability test (after Rahardjo, 2014a)

**Laboratory Tests**

Laboratory tests were performed on undisturbed samples obtained from site investigations of all slopes in batches 1, 2 and 3. The laboratory tests comprised of index properties tests, saturated and unsaturated permeability, soil-water characteristic curves (SWCC), saturated and unsaturated triaxial tests. The testing program provided information on the basic soil properties and the engineering properties of different soil layers.

**Standard Laboratory Test**

- **Index Properties**

  The index properties tests were carried out in accordance to ASTM D4318–00 (2000) are as follows:

  1. Natural water content tests.
  2. Specific gravity tests (to obtain $G_s$)
  3. Atterberg limits tests (to obtain LL, PL and PI)
  4. Wet sieve analyses
  5. Mechanical sieve analyses
  6. Hydrometer tests
The soil samples were classified under the Unified Soil Classification System (USCS) using the information from the index properties tests. Grain-size distribution data were obtained from a combination of mechanical sieve analyses and hydrometer tests (for fine particles or particle sizes < 75 μm). Wet sieving was performed to separate fine particles that stick into coarse particles.

Grain-size distributions of residual soils from JF, BTG and OA slopes in this project are plotted in Figures 4 to 6. Typical grain-size distribution data were calculated based on the average percentage passing of each particle diameter. The minimum percentage of fine particles (i.e. particle size < 75 μm) for residual soils from JF, BTG and OA is 23%, 30% and 23%, respectively. The maximum percentage of fine particles for residual soils from JF, BTG and OA are similar, which is about 95%. Typical percentages of fine particles for residual soils from JF, BTG and OA are 55%, 62% and 60%, respectively.

![Figure 4: Grain size distribution of residual soil from sedimentary Jurong Formation (Rahardjo et al., 2012f)](image1)

![Figure 5: Grain size distribution of residual soil from Bukit Timah Granite (Rahardjo et al., 2012f)](image2)
In general, residual soils from BTG and OA are coarser than residual soils from JF. The distributions of fine particles for residual soils from BTG and OA are similar with only small percentages of gravel observed in the grain-size distribution of these two residual soils. However, a higher percentage of clay particles and a lower percentage of silt particles are found in residual soils from OA as compared with those from BTG. Some residual soils from BTG also have grain-size distribution with bimodal characteristics (gap-graded soils).

![Grain size distribution of residual soil from Old Alluvium (Rahardjo et al., 2012f)](image)

Figure 6: Grain size distribution of residual soil from Old Alluvium (Rahardjo et al., 2012f)

- **Saturated permeability and shear strength**

  Saturated permeability tests were carried out by NTU and Tritech Engineering & Testing Pte Ltd using different methods. In the Tritech laboratory, the permeability tests were performed using falling head tests in accordance with the procedure described in the “Manual of Soil Laboratory Testing-Volume 2” (Head, 1986). In NTU laboratory, permeability tests were performed using a triaxial cell with two back-pressure systems as described in the “Manual of Soil Laboratory Testing-Volume 3”, (Head, 1986). Both methods were suitable for soils of intermediate and low permeability, such as silt and clay with permeabilities less than 10⁻⁴ m/s. The saturated triaxial tests in NTU allowed the measurement of \( k_s \) (i.e. coefficients of permeability) under various confining pressures.

  Shear strength parameters (i.e. \( c' \) and \( \phi' \)) were obtained from consolidated undrained (CU) saturated triaxial tests with pore-water pressure measurements at NTU and Tritech Engineering & Testing Pte Ltd. Multistage saturated triaxial tests were performed in accordance with ASTM D7181-11 (2009). Two failure criteria were adopted following Head (1986): (1) Flattening out of the stress/strain curve and (2) Maximum principal stress ratio \((\sigma_1' / \sigma_3')\) had been reached.
Unsaturated Laboratory Test

- Soil-Water Characteristic Curves

Soil-water characteristic curves (SWCCs) are an important soil property in unsaturated soil mechanics. The SWCC relates the amount of water in the soil to matric suction \((u_a-u_w)\) of the soil. Matric suction is applied to the specimen using the axis-translation technique (Hilf, 1956). The magnitude of the matric suction is the difference between the air pressure and water pressure applied to the specimen. The SWCC was determined by combining the results from Tempe cell tests, Pressure Plate tests and salt solution equilibration test method. SWCCs were obtained for soil samples from seven slopes in batch 1, five slopes in batch 2 and five slopes in batch 3.

The maximum air-entry values (AEVs) of the ceramic disk in a Tempe cell was 1 bar (100 kPa) while the Pressure Plate apparatus had an air-entry value of 15 bars (1500 kPa). Therefore, the test using the Tempe cell was performed up to matric suction of 100 kPa, while the Pressure Plate was used for the application of matric suction up to 1500 kPa. For suction values greater than 1500 kPa, the salt solution equilibration test was used. Schematic diagrams for each apparatus are shown in Figures 7 to 9. The details and procedures of SWCC tests can be seen in Fredlund et al. (2012) and Fredlund and Rahardjo (1993).

![Figure 7: Schematic diagram of Tempe cell (Rahardjo et al., 2007c)](image)

![Figure 8: Schematic diagram of pressure plate (Rahardjo et al., 2007e)](image)
The SWCC of residual soils from JF, BTG and OA in this project were compiled with the SWCC data from Agus et al. (2001). The normalized drying SWCC data for residual soils from JF, BTG and OA are shown in Figures 10, 12 and 14, respectively. A typical SWCC was obtained by taking a mean value of volumetric water content for each matric suction within the upper bound and lower bound of SWCC for residual soils in Singapore. The SWCC parameters (i.e., AEV, residual suction, residual water content) for the residual soils from JF, BTG and OA were determined using Zhai and Rahardjo (2012) equations.

The wetting curve of SWCC is crucial since most of the geotechnical problems, such as slope failure are induced by rainfall infiltration that generally causes a wetting of the soil. Therefore, it is also important to carry out SWCC tests under the wetting process. The measurement of the wetting curve is troublesome and time-consuming. Therefore, the average, upper and lower bound limits for the wetting portion of the SWCC were estimated from the drying curve using Pham et al. (2005) equation. The boundaries of the drying and wetting curves for residual soils in Singapore are presented in Figures 11, 13 and 15.
The SWCC parameters for residual soils in Singapore are summarized in Table 4. It can be observed that the ranges of air-entry values for residual soils from JF and OA are wider than those for residual soils from BTG. The difference can be attributed to the greater variation of pore sizes of residual soils from JF and OA as compared to that of residual soils from BTG. The typical air-entry values of residual soils from BTG and OA are lower than that of residual soils from JF indicating the pore sizes of residual soils from BTG and OA are bigger than that observed in residual soils from JF. This difference is also supported by the lower saturated water content and the steeper slope of SWCC of residual soils from BTG and OA than those of residual soils from JF. Table 4 also shows that the typical residual water content and the suction of residual soils from BTG and JF are lower than those of residual soils from JF.
Figure 13: Drying and wetting soil-water characteristic curve of residual soil from Bukit Timah Granite (Rahardjo et al., 2012f)

Figure 14: Compilation of drying soil-water characteristic curve of residual soils from Old Alluvium (Rahardjo et al., 2012f)

Figure 15: Drying and wetting soil-water characteristic curve of residual soil from Old Alluvium (Rahardjo et al., 2012f)
Table 4: Range of SWCC properties of residual soil in Singapore calculated using Zhai and Rahardjo (2012) equation

<table>
<thead>
<tr>
<th>Soil</th>
<th>Air-entry value (kPa)</th>
<th>Saturated water content</th>
<th>Residual suction (kPa)</th>
<th>Residual water content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sedimentary Jurong Formation</td>
<td>1 to 116</td>
<td>0.3 to 0.60</td>
<td>1500 to 18000</td>
<td>0.025 to 0.100</td>
</tr>
<tr>
<td>Bukit Timah Granite</td>
<td>0.8 to 25</td>
<td>0.21 to 0.61</td>
<td>106 to 12000</td>
<td>0.015 to 0.098</td>
</tr>
<tr>
<td>Old Alluvium</td>
<td>5 to 40</td>
<td>0.24 to 0.5</td>
<td>42 to 12000</td>
<td>0.009 to 0.098</td>
</tr>
</tbody>
</table>

- **Unsaturated Triaxial Test**

Unsaturated shear strength parameters ($\phi^b$) of the investigated soils from JF, BTG and OA were obtained from unsaturated triaxial tests at NTU. The unsaturated triaxial tests were only conducted on soil samples from seven slopes in batch 1, five slopes in batch 2 and five slopes in batch 3.

The modified triaxial apparatus (Ong, 1999) was used for consolidated drained (CD) test on unsaturated soils (Figure 16). The triaxial apparatus is capable of controlling and/or measuring pore-air and pore-water pressures in the soil specimen independently using the axis-translation technique. This allows any desired matric suction to be applied to the soil in the laboratory. Details and procedures pertaining to performing unsaturated triaxial tests can be obtained from Fredlund et al. (2012) and Fredlund and Rahardjo (1993).

![Figure 16: Schematic diagram of modified triaxial cell for unsaturated triaxial tests (Ong, 1999)](image)

The results of the unsaturated triaxial CD tests were interpreted using Mohr circles. The failure envelope sloping at an angle of $\phi'$ was drawn tangent to the Mohr circles at failure. The failure envelope intersected the shear strength versus matric suction plane at a cohesion intercept, $c$. $\phi'$ and $c'$ were determined from the
saturated CU tests described previously. The cohesion intercepts obtained at various matric suctions were combined to give the $\phi^b$ angle. The details and procedures in analyzing results from the unsaturated triaxial tests for residual soil slopes in Singapore can be found in Rahardjo et al. (2009b)

- **Unsaturated Permeability Test**

The unsaturated coefficient of permeability was determined using an indirect methodology. A statistical model forms the basis for the common indirect methods used to predict the permeability function based on the saturated coefficient of permeability, $k_s$, and the soil-water characteristic curve (Marshall, 1956; Millington and Quirk, 1957, 1961; Kunze, 1968; Green and Corey, 1971) as described in Fredlund and Rahardjo (1993). The predicted permeability data can then be fitted using the best fit equation as suggested by Leong and Rahardjo (1997b). In this study, the drying and wetting permeability functions were both considered.

Figures 17 to 19 show the permeability functions of residual soils from JF, BTG and OA. The typical permeability function was obtained by taking a mean value of permeability for each matric suction for residual soils in Singapore. The mean value of the permeability function was then fitted using the Leong and Rahardjo equation (1997b).

![Figure 17: Permeability functions of residual soils from sedimentary Jurong Formation (Rahardjo et al., 2012f)](image)

**Case Study**

In this section, the laboratory results from three sites which are Slope 1-3, Slope 1-7 and Slope 3-7 representing JF, BTG and OA, respectively are presented and discussed. Simplified soil profiles of residual soil slope from the three sites are presented in Figures 20 to 22. Both soil layers at Slope 1-3 (Figure 20) can be classified as clayey sand with $c'$ varying from 0 kPa to 4 kPa, $\phi'$ varying from 33° to 36° and $\phi^b$ varying from 25° to 26.5°. All layers at Slope 1-7 (Figure 21) can be
classified as silty sand with $c'$ varying from 0 kPa to 9 kPa and $\phi^b$ varying from 21° to 26°. $\phi'$ of all layers at Slope 1-7 are around 33°. Soil layers at Slope 3-7 (Figure 22) are uniform and can be classified as silty sand with $c'$ = 18 kPa, $\phi'$ = 35° and $\phi^b$ = 30°.

Figure 18: Permeability functions of residual soils from Bukit Timah Granite (Rahardjo et al., 2012f)

Figure 19: Permeability functions of residual soils from Old Alluvium (Rahardjo et al., 2012f)

Figure 20: Soil profile of residual soil slope from sedimentary Jurong Formation at Slope 1-3 (Rahardjo et al., 2011 c)
Figure 21: Soil profile of residual soil slope from Bukit Timah Granite at Slope 1-7 (Rahardjo et al., 2011 c)

Figure 22: Soil profile of residual soil slope from Old Alluvium at Slope 3-7 (Rahardjo, 2014)

Index properties of residual soils from Slope 1-3, Slope 1-7 and Slope 3-7 are presented in Figures 23 to 25, respectively.

Figure 23: Properties of residual soil from sedimentary Jurong Formation at Slope 1-3
Figure 24: Properties of residual soil from Bukit Timah Granite at Slope 1-7

Figure 25: Properties of residual soil from Old Alluvium at Slope 3-7

Figure 26: Soil-water characteristic curves (SWCC) for residual soils from sedimentary Jurong Formation at Slope 1-3 and Bukit Timah Granite at Slope 1-7

The drying soil-water characteristic curves (SWCCs) of specimens obtained from different layers at Slope 1-3, Slope 1-7 and Slope 3-7 are presented in Figure 26. SWCC for the residual soil from the JF at Slope 1-3 has a lower
saturated volumetric water content, a higher air-entry value, a gentler slope and a
lower residual water content than those of SWCC for the residual soil from BTG
at Slope 1-7 and OA at Slope 3-7. Permeability functions for the residual soils
from the three sites were estimated using the statistical method and are presented in
Figure 27.

Figure 27: Permeability functions for residual soil slope from sedimentary Jurong
Formation at Slope 1-3, Bukit Timah Granite at Slope 1-7 and Old Alluvium at Slope
3-7

Figures 28 to 30 show the deviator stress and water volume change versus
axial strain curves associated with the multistage Consolidated Drained (CD)
unsaturated triaxial tests. The stress-strain curves of the residual soil at Slope 1-3,
Slope 1-7 and Slope 3-7 showed a ductile behaviour throughout the shearing
process. The deviator stress increased with the increase in the axial strain,
whereas the water volume change decreased with the increase in the axial strain.
In addition, the peak deviator stress, the stiffness of the specimen and the water
volume change of the specimen increased with the increase of the confining
pressure.

Figure 28: (a) deviator stress and (b) water volume change versus axial strain for CD
unsaturated triaxial test under 50 and 100 kPa confining pressures for residual soils
at Slope 1-3
Figures 28 to 30 indicate that residual soil from BTG have higher deviator stress than those from JF and OA at a given matric suction for the same confining pressure. This could be attributed to the larger particle sizes of residual soil from BTG as compared to those from JF and OA. Figures 28 to 30 also show that the rate of increase in deviator stress for all residual soils in unsaturated condition decreased with the increase in matric suction. The observed behaviour was due to the fact that, as air replaced some of water within large pores, water only existed in smaller pores and formed water menisci between particle contact points of residual soils. As a result, the effectiveness of matric suction in increasing shear strength of the soil started to reduce.

The peaks of deviator stress from stress-strain curves obtained from the unsaturated triaxial tests were used to draw the Mohr circles at failure (Figures 32 to 34). Then the three-dimensional extended Mohr-Coulomb failure envelope (Figure 31) was used to generate the unsaturated shear strength (Fredlund et al.,
of residual soil. The unsaturated shear strength data from laboratory tests were fitted using Goh et al. (2010) equation (Equations 1 and 2). The \( \phi^b \) angles of residual soils from JF (30\(^o\)), BTG (40\(^o\)) and OA (35\(^o\)) were observed to be constant for matric suctions ranging between zero and the AEV (Figure 35). The \( \phi^b \) angles started to decrease non-linearly at suction beyond the AEV.

\[
\tau = c' + \left( \sigma - u_a \right) \tan \phi' + \left( u_a - u_w \right) \tan \phi^b
\]

for \((u_a - u_w) \leq \text{AEV}\) \hspace{1cm} (1)

\[
\tau = c' + \left[ \left( \sigma - u_a \right) + \text{AEV} \right] \tan \phi' + \left[ \left( u_a - u_w \right) - \text{AEV} \right] b \Theta^k \tan \phi^b
\]

for \((u_a - u_w) > \text{AEV}\) \hspace{1cm} (2)

where: \(\tau = \text{shear strength; } (\sigma - u_a) = \text{net normal stress; } \phi' = \text{effective friction angle; } \phi^b = \text{angle indicating the increase in shear strength due to the increase in suction } (\phi^b = \phi' \text{ for } (u_w - u_a) \leq \text{AEV}); (u_w - u_a) = \text{matric suction; } \Theta = \text{normalized water content } (\theta_w/\theta_s); b, k = \text{fitting parameters.}

Figure 31: Extended Mohr-Coulomb failure envelope for unsaturated soil

Figure 32: Typical unsaturated shear strength of residual soil from sedimentary Jurong Formation obtained from multistage triaxial test at confining pressure of 25 and 50 kPa
Shear Stress, \( \tau \) (kPa)

Matric Suction (\( u_a - u_w \)) (kPa)

Net Normal Stress \( \left( \sigma - u_a \right) \) (kPa)

\( \phi' = 36^o \)
\( c_1 = 24 \text{ kPa} \)
\( c_2 = 28 \text{ kPa} \)
\( c_3 = 31 \text{ kPa} \)

Air-entry value = 20

\( \phi' = 30^o \)
\( c_1 = 18 \text{ kPa} \)
\( c_2 = 40 \text{ kPa} \)
\( c_3 = 43 \text{ kPa} \)

Air-entry value = 15

\( \phi' = 30^o \)

Figure 33: Typical unsaturated shear strength of residual soil from Bukit Timah Granite obtained from multistage triaxial test at confining pressure of 25 and 50 kPa

Figure 34: Typical unsaturated shear strength of residual soil from Old Alluvium obtained from multistage triaxial test at confining pressure of 25 and 50 kPa

Figure 35: The experimental data of the unsaturated shear strength of residual soils from JF, BTG and OA as fitted with Goh et al. (2010) equation

Variability of Residual Soil Properties

Index Properties of Soil

The typical, upper and lower bound of the distribution of soil particles for various depths of residual soils from JF, BTG and OA are plotted and shown in Figure 36. Figure 36a shows that the typical percentage of sand and fine particles
for residual soils from JF increases linearly with depth from the ground surface (silty clay) until 7 m depth (clayey sand). However, the percentages of sand and fine particles for residual soils from JF are about constant for depths from 7 m to 14 m. Figure 36a also shows that the typical residual soil from JF is classified as a coarse-grained soil (i.e., percentage of sand is higher than 50%) for depths greater than 7 m.

The percentage of sand increases non-linearly with depth in a similar trend with the percentage of fine particles for residual soil from BTG (Figure 36b). The percentage of sand for residual soil from BTG increases significantly from ground surface until 2 m depth and starts to increase gradually at greater depths. Similar to residual soils from JF, typical residual soils from BTG are also classified as a coarse-grained soil for depths greater than 7 m. However, the percentage of sand for residual soils from BTG increases at depths greater than 7 m. Therefore, typical residual soils from BTG at various depths are coarser than typical residual soils from JF.

Figure 36: Distribution of soil particle with depth for residual soil from (a) sedimentary Jurong Formation, (b) Bukit Timah Granite and (c) Old Alluvium

The percentage of sand also increases non-linearly for residual soils from OA (Figure 36c). However, the percentage of sand increases more drastically with depth as compared to the percentage of clay. On the other hand, the percentage of silt for residual soils from OA decreases with depth. As depicted in Figure 36c, the percentages of sand and clay for residual soils from OA are higher at greater depths. In addition, typical residual soils from OA are classified as a fine-grained soil from ground surface until 14 m depth. Figure 36 also shows that the percentage of clay in residual soil from JF is higher as compared to that in residual soils from OA, indicating that the residual soil from JF is more weathered as compared to that from OA. In addition, that the percentage of clay for residual
soil from OA is higher than that from BTG, indicating that the residual soil from OA is more weathered as compared to that from BTG.

The coefficient of variation, COV of the inherent variability (COV$_w$) of soil particle distribution is plotted versus the mean of soil particle distribution (percentage of sand and clay) as shown in Figure 37. The COV$_w$ of all residual soils decreases with increasing mean values of soil particle distribution. Typical ranges of COV$_w$ for residual soils from JF, BTG and OA are 3-47 %, 5-34 % and 10-38 %, respectively. The range of COV$_w$ for residual soils from BTG is narrower than that for residual soils from JF. This indicates that the particle size distribution of residual soils from BTG is more uniform than that from JF. This behaviour occurs because residual soils from BTG were formed only from one type of rock, granite. On the other hand, residual soils from JF were formed from different types of rock (i.e. mudstone, siltstone, sandstone, shale and conglomerate). Figure 37 also shows that the COV$_w$ for residual soils from OA is narrower than that for residual soils from BTG. This indicates that the particle size distribution of residual soils from OA is more uniform than that from BTG.

![Figure 37: Coefficient of variation of inherent variability (COV$_w$) of soil particle distribution with depth for residual soil from sedimentary Jurong Formation, Bukit Timah Granite and Old Alluvium](image)

The distribution of liquid limit (LL), plastic limit (PL) and natural water content ($w_n$) of residual soils at various depths were compiled and plotted in Figures 38 to 40. Figures 38 to 40 show that typical LL, PL and $w_n$ for residual soils from JF, BTG and OA decreased non-linearly with depth from the ground surface until 14m depth. These can be understood since the percentages of clay for residual soils from JF, BTG and OA decreased with depth from the ground surface until 14m depth.
Figure 38: Distribution of liquid limit with depth for residual soil from (a) Sedimentary Jurong Formation, (b) Bukit Timah Granite and (c) Old Alluvium

Figures 38a, 39a and 40a show that the boundaries of LL, PL and $w_n$ for residual soils from JF varied with depth. These trends were not observed in residual soils from BTG and OA (Figures 38b, 38c, 39b, 39c, 40b and 40c). These are related to the fact that the boundary of soil particle distribution for residual soils from JF also varies with depth. Figures 39 and 40 show that natural water contents of residual soils from JF, BTG and OA were close to the plastic limits throughout the depth indicating the unsaturated condition of the residual soils (Fredlund and Rahardjo, 1993).

Figure 39: Distribution of plastic limit with depth for residual soil from (a) Sedimentary Jurong Formation, (b) Bukit Timah Granite and (c) Old Alluvium
The variations of COV\(_w\) of LL and PL for residual soils from JF, BTG and OA are plotted and shown in Figure 40. No trends are observed for the mean of LL and PL in Figure 41. The mean values of LL for residual soils from JF, BTG and OA vary from 41% to 45%, from 41% to 51% and from 56% to 58%, respectively. The mean values of PL for residual soils from JF, BTG and OA vary from 20% to 22%, from 31% to 37% and from 36% to 42%, respectively. The typical ranges of COV\(_w\) of LL (5% to 33%) and PL (17% to 30%) for residual soils from JF are wider than the typical ranges of COV\(_w\) of LL (15% to 32%) and PL (24% to 31%) for residual soils from BTG. These COV\(_w\) indicate that the variabilities of LL and PL, which correspond to the boundary of LL and PL, are higher for residual soils from JF than those for residual soils from BTG. The typical ranges of COV\(_w\) of LL and PL for residual soils from JF (LL = 5% to 33%, PL = 17% to 31%), BTG (LL = 15% to 32%, PL = 24% to 32%) and OA (LL = 13% to 20%, PL = 21% to 32%) are in agreement with the typical ranges of COV\(_w\) of LL (7% to 39%) and PL (6% to 34%) for fine-grained soils as shown in Phoon and Kulhawy (1999).

Weathering processes of rock formation in Singapore result in the porous structure of residual soil. Therefore, the void ratio of residual soils in Singapore varies with depth. The variations of void ratio with depth for residual soils in this project and plotted in Figure 42. It was observed that typical void ratio of residual soils from JF, BTG and OA is high near ground surface and decreases non-linearly with depth. As a result, total density increases with depth since water and air occupy more space in the upper part of residual soils from JF, BTG and OA.
As shown in Figure 42, the decreasing trend of void ratio of residual soils from JF is not as obvious as that of residual soils from BTG, while the void ratio of residual soils from BTG decreases in a less significant manner as compared to that of residual soils from OA. This corresponds to the decreasing trends of soil particle distribution of residual soils from JF, BTG and OA. Figure 42 also shows that the range in void ratio for residual soils from OA (0.47 to 0.85) is higher than that for residual soils from JF (0.3 to 0.78) and BTG (0.3 to 0.75). These can be attributed to the higher percentage of clay for residual soils from OA than that for residual soils from JF and BTG.

The variations of the COV of void ratio for residual soils from JF, BTG and OA are plotted in Figure 43. No trends are observed for the mean of void ratio in Figure 43. The mean values of void ratio for residual soils from BTG and OA vary from 0.49 to 0.6 and from 0.5 to 0.8, respectively. The mean value of
void ratio for residual soils from JF is relatively constant around 0.52. The typical ranges of COV of void ratio for residual soils from JF, BTG and OA are 15 % to 44 %, 16 % to 33 % and 3 % to 10 %, respectively. These indicate that the variabilities of void ratio for residual soils from BTG are lower than those for residual soils from JF, but are higher than those for residual soils from OA.

![Figure 43: Coefficient of variation (COV) of void ratio for residual soil from Jurong Formation, Bukit Timah Granite and Old Alluvium](image)

**Shear Strength of Soil**

It was observed that the weathering process resulted in the variation of effective cohesion ($c'$), effective friction angle ($\phi'$) and $\phi^b$ angle with depth for residual soils in Singapore. Therefore, it is important to quantify the shear strength properties of residual soils at various depths. The typical and the boundary values of the $c'$, $\phi'$ and $\phi^b$ are plotted in Figures 44, 46 and 48, respectively. Typical $c'$ of residual soils from JF decreases with depth although the decrease in $c'$ for residual soils from JF is not as significant as that of residual soils from BTG and OA (Figure 45). This occurs since the percentages of fine particles for residual soils from JF, BTG and OA also decrease with depth. The typical $c'$ value of residual soils from BTG is lower than that of residual soils from JF since residual soils from BTG are coarser than residual soils from JF.

The relationships between COV of $c'$ for residual soils from JF, BTG and OA are shown in Figure 45. The mean $c'$ values for residual soils from JF, BTG and OA vary from 11 kPa to 14 kPa, 8 kPa to 12.5 kPa and 18 kPa to 24 kPa, respectively. The COV of $c'$ for residual soils from JF is wider than that for residual soils from BTG and OA. This can happen because the degree of weathering for residual soils from JF is more variable than that for residual soils
from BTG and OA. Figure 45 also shows that the range of COV_w of c’ for residual soils from BTG is similar to that for residual soils from OA.

![Figure 44: Distribution of effective cohesion with depth for residual soil from (a) sedimentary Jurong Formation, (b) Bukit Timah Granite and (c) Old Alluvium](image)

![Figure 45: Coefficient of variation (COV_w) of inherent variability of effective cohesion for residual soil from Jurong Formation, Bukit Timah Granite and Old Alluvium](image)

The effective friction angle (ϕ’) of residual soils from JF, BTG and OA increases with depth (Figure 46) because the percentage of sand increases with depth (Figure 35). The increasing trend of ϕ’ for residual soils from JF is similar to that for residual soils from BTG and OA. The observed behaviour can be attributed to the fact that the effective friction angle is affected by texture, size and distribution of particles in soil. The typical mean line, the upper and lower boundaries of the effective friction angle for residual soils from BTG are higher
than those for residual soils from JF. These are due to the fact that the particle sizes of residual soil from BTG are larger than those of residual soils from JF. Figure 46 also shows that the particle size of residual soil from OA is similar to that of residual soil from BTG since the typical mean line and boundaries of residual soils from OA is similar to that of residual soils from BTG.

Figure 46 shows that the range of mean $\phi'$ for residual soils from JF (28° to 35°) is lower than that for residual soils from BTG (33° to 37°). This suggests that the particle size of residual soils from JF is finer than that of residual soils from BTG. On the other hand, the range of mean values of $\phi'$ for residual soils from BTG is similar to that for residual soils from OA (35° to 37°). This indicates that there is a similarity in the majority of particle size distribution between residual soils from BTG and those from OA. The range of COV_w for residual soils from JF is much wider than that for residual soils from BTG (Figure 47). This verifies the fact that the shear strength of residual soils from JF in a saturated condition is more diverse than that of residual soils from BTG. The ranges of COV_w of $\phi'$ for residual soils from JF (4° to 14°) are in agreement with the ranges of COV_w of $\phi'$ for silt and clay (4° to 12°) as observed in Phoon and Kulhawy (1999). On the other hand, the ranges of COV_w of $\phi'$ for residual soils from BTG (8° to 15°) and OA (7° to 15°) are in agreement with the ranges of COV_w of $\phi'$ for sand (5° to 11°) as shown in Phoon and Kulhawy (1999).

Figure 46 shows that the range of mean $\phi'$ for residual soils from JF (28° to 35°) is lower than that for residual soils from BTG (33° to 37°). This suggests that the particle size of residual soils from JF is finer than that of residual soils from BTG. On the other hand, the range of mean values of $\phi'$ for residual soils from BTG is similar to that for residual soils from OA (35° to 37°). This indicates that there is a similarity in the majority of particle size distribution between residual soils from BTG and those from OA. The range of COV_w for residual soils from JF is much wider than that for residual soils from BTG (Figure 47). This verifies the fact that the shear strength of residual soils from JF in a saturated condition is more diverse than that of residual soils from BTG. The ranges of COV_w of $\phi'$ for residual soils from JF (4° to 14°) are in agreement with the ranges of COV_w of $\phi'$ for silt and clay (4° to 12°) as observed in Phoon and Kulhawy (1999). On the other hand, the ranges of COV_w of $\phi'$ for residual soils from BTG (8° to 15°) and OA (7° to 15°) are in agreement with the ranges of COV_w of $\phi'$ for sand (5° to 11°) as shown in Phoon and Kulhawy (1999).

Figure 46: Distribution of effective friction angle with depth for residual soil from (a) sedimentary Jurong Formation, (b) Bukit Timah Granite and (c) Old Alluvium
Figure 47: Coefficient of variation of inherent variability (COV_x) of effective friction angle for residual soil from Jurong Formation, Bukit Timah Granite and Old Alluvium

The variations of typical $\phi^b$ angle with depth for residual soils (Figure 48) show a similar trend with those observed in the variations of $\phi'$ (Figure 47). Typical $\phi^b$ angle for all residual soils increases with depth due to the higher contents of coarser particles in the greater depths. Typical variations of $\phi^b$ angle with depth for all residual soils are similar. The mean value of $\phi^b$ for residual soils from JF, BTG and OA varies from 28° to 32.5°. The range of COV_x of $\phi^b$ for residual soils from JF is wider than that for residual soils from BTG (Figure 49). This can be attributed to the higher variability of degree of weathering of residual soils from JF as compared to that of residual soils from BTG.

Figure 48: Distribution of $\phi^b$ angle with depth for residual soil from a. sedimentary Jurong Formation and b. Bukit Timah Granite and c. Old Alluvium
Figure 49: Coefficient of variation (COV) of $\phi_b$ angle for residual soil from Jurong Formation, Bukit Timah Granite and Old Alluvium

**Field Instrumentation**

Field instrumentation was carried out at each slope to monitor the effect of rainfall on the pore-water pressures. The sensors used in the field instrumentation consisted of piezometers, tensiometers, rain gauges, depth transmitters or vibrating wire, pressure transducers, data acquisition systems, power supply systems and alert systems. Typical layout of instrumentation for on-line monitoring system is illustrated in Figure 50.

![Typical layout of instrumentation for on-line monitoring system](image)

Figure 50: Typical layout of instrumentation for on-line monitoring system (Rahardjo et al., 2008a)

**Standard Instrumentation**

- **Piezometers**

Casagrande piezometers were used for groundwater table monitoring at some completed boreholes in the slope because they consist of economical
components, simple to read and have long-term reliability. The details of Casagrande piezometer are shown in Figure 51.

![Casagrande piezometer diagram]

**Figure 51: Details of Casagrande piezometer**

- **Inclinometer**

  Some of the completed boreholes from each slope were used for the installation of inclinometers which were used to monitor the deformation of the investigated slopes.

**Advanced Instrumentation**

- **Tensiometers**

  In this research, tensiometers were installed at two slopes near Slope 1-3 and Slope 1-7. This instrument is capable of measuring negative pore-water pressures (matric suctions) in the soil (slope) (Rahardjo et al., 2007d). The tensiometer used for on-line monitoring at Slope 1-3 and Slope 1-7 were Soilmoisture™ Model 2725 Jet-fill tensiometers (Soilmoisture, 1986), as shown in Figure 52.

  Regular maintenance of the tensiometers was required to remove accumulated air in the tensiometer, and to refill the Jet-fill cup, if necessary. The accumulation of air was a result of the diffusion of air through the ceramic tip or cavitation of water inside the tensiometer body.

- **Rain gauges**

  A rainfall gauge was installed to determine the intensity of rainfall on the site. A tipping bucket rainfall gauge was chosen to collect the rainfall data as shown in Figure 53. Whenever the tipping bucket tipped, the switch closed an electric circuit temporarily and a data acquisition system (DAS) recorded the current surge. The process repeated continuously until the end of rainfall. The
rainfall intensity was calculated from the water volume of the tipping bucket corresponding to each tip.

Figure 52: Jet-fill tensiometer with transducer (Rahardjo et al., 2008a)

Figure 53: Tipping bucket rainfall gauge (Rahardjo et al., 2008a)

On-line Monitoring System

- Depth transmitters or vibrating wire

  The piezometers installed at Slope 1-3, Slope 1-7 and Slope 3-7 were fitted with depth transmitters to allow for automated data acquisition. The system allowed the real time observation of the ground water table locations and groundwater table fluctuations during wet and dry periods. The depth transmitter took a reading by converting pore-water pressure to a frequency signal through a diaphragm, a tensioned steel wire, and an electromagnetic coil. It allowed for accurate monitoring of ground water level changes in response to rainfall.

- Pressure Transducers

  The pressure transducer was connected to a data acquisition system (DAS) to record pore-water pressure in real time for both positive and negative ranges:
from -100 to 75 kPa relative to atmospheric pressure. A pressure correction was required to convert the pore-water pressure measured by the transducer, which was located at the top of the instrument, to the actual pore-water pressure at the tensiometer tip, which was located at the bottom of the tensiometer.

- **Data Acquisition Systems**

  The data acquisition system (DAS) was installed to allow for the automatic collection of pore-water pressures, groundwater levels, and rainfall data which were recorded from tensiometers, piezometers and rain gauge, respectively. The DAS was programmed to be able to record the time and date of every rain gauge tip. The number of tips that have occurred was tallied to measure the rainfall intensity every hour.

  Data were required to be collected at different time intervals to optimize the quality and quantity of the recorded data. Different time intervals were used to collect data depending on the climatic conditions in the field. During rainfall, the data were collected at shorter time intervals to record the changing field conditions accurately. All transducers from the instrumentations were connected to the same power supply and data logger. A warning system was also installed in the DAS.

- **Power Supply Systems**

  The power supply system may vary depending on the location. Since the location of the instrumentation was far away from an electrical power supply, a self-sufficient power supply was required. Two lead cell batteries were provided as power supplies for this system because it was the cheapest power supply and it could supply sufficient power for the on-line monitoring with the required interval of instrumentation reading. In other cases, a photovoltaic power supply system is usually selected.

**Case Study**

- **Instrumentation Layout**

  Three case studies were carried out on three residual soil slopes at Slope 1-3, Slope 1-7 and Slope 3-7. The schematic diagram of instrumentation layout for the residual soil slopes is shown in Figure 54.

- **Relationship between Rainfall Intensity and Pore-water Pressure**

  The online and real time monitoring programs of instruments at Slope 1-3 and Slope 1-7 were carried out from July 2007 to July 2009. While the period for Slope 3-7 was from June 2010 to January 2011. Figure 55 shows the pore-water
pressures at various measuring depths for a row of tensiometers installed near the toe of the Slope 1-3 as monitored from September to November 2008. Figure 56 shows the pore-water pressures at various measuring depths for a row of tensiometers installed near the toe of the Slope 1-7 as monitored from February to May 2008. Similarly, the pore-water pressures at various measuring depths for a row of tensiometers installed near the toe of the Slope 3-7 from June 2010 to January 2011 are shown in Figure 57.

Figure 54: Generalized geological map of Singapore and schematic diagram of relative position and arrangement of field instrumentation for residual soil slope at Slope 1-3, Slope 1-7 and Slope 3-7 (Rahardjo et al., 2011d)

Figure 55: Pore-water pressure readings at various depths of residual soil slope at Slope 1-3 (monitoring period: September to November 2008)
Figure 56: Pore-water pressure readings at various depths of residual soil slope at Slope 1-7 (monitoring period: February to May 2008)

Figure 57: Pore-water pressure readings at various depths of residual soil slope at Slope 3-7 (monitoring period: June 2010 to January 2011)

- Contour of Pore-water Pressure and Total Head

The pore-water pressure and total head contours for Slope 1-3 before and at the end of rainfall are presented in Figures 58 and 59. There was the presence of significant matric suctions near the ground surface during the dry period which affected the stability of the slope (Figure 58a). It can be seen in Figure 58b that moisture migration occurred within the slope even during the dry period when water moved upward across the slope surface as evaporation and also along the slope. Figure 59a shows that matric suction decreased near the ground surface, while total head distribution in Figure 59b illustrates that water flowed across the slope surface and downward through the slope during the wet period.
The pore-water pressure and total head contours based on measured data obtained from the field at Slope 1-7 before and at the end of rainfall are presented in Figures 60 and 61. A significant negative pore-water pressure near the ground surface during the dry period is illustrated in Figure 60a which could affect the stability of the slope. The moisture movement was not only across the slope, but also in the upward direction on the slope (Figure 60b). During the wet period, the matric suction would decrease due to rainfall (Figure 61a) and, at the same time, the total head contours show that water flowed along the slope and percolated downwards through the slope as shown in Figure 61b.
Variation of Groundwater Table Positions for JF, BTG and OA

The investigated residual soil slopes in this project were instrumented with at least two piezometers to monitor the position of groundwater table. Manual monitoring of the Casagrande piezometers for all slopes was carried out over a two-year period (June 2006 to September 2008). Variations of groundwater table positions for the driest and wettest periods are shown in Figures 62 to 64 in the residual soil slopes from JF, BTG, OA, respectively. The average positions of groundwater table, representing the typical position in residual soil slopes in Singapore, were calculated based on the minimum position of groundwater table during the dry period and the maximum position of groundwater table during the wet period. The symbol $\ell$ in Figures 62 to 64 represents the distance of the piezometers (P1 and P2) measured from the crest of the slopes, while the symbol $h$ corresponds to the depth of groundwater table from the slope surfaces. Slope vertical heights and slope horizontal lengths were denoted as $H$ and $L$, respectively.

![Figure 61: (a) Pore-water pressure and (b) total head contour of the Slope 1-7 site on 7 May 2008 after rainfall (Rahardjo et al., 2009c)](image)

![Figure 62: Variation of ground water table position for residual soil slopes from sedimentary Jurong Formation (Rahardjo et al., 2010b)](image)
The driest period
\[ h/H = -0.581 \left( \frac{l}{L} \right) + 0.823 \quad R^2=0.83 \]

The wettest period
\[ h/H = -0.599 \left( \frac{l}{L} \right) + 0.648 \quad R^2=0.81 \]

Average
\[ h/H = -0.59 \left( \frac{l}{L} \right) + 0.736 \]

Figure 63: Variation of ground water table position for residual soil slopes from Bukit Timah Granite (Rahardjo et al., 2010b)

The driest period
\[ h/H = -0.65 \left( \frac{l}{L} \right) + 0.757 \quad R^2=0.91 \]

The wettest period
\[ h/H = -0.65 \left( \frac{l}{L} \right) + 0.547 \quad R^2=0.92 \]

Average
\[ h/H = -0.65 \left( \frac{l}{L} \right) + 0.659 \]

Figure 64: Variation of ground water table position for residual soil slopes from Old Alluvium (Rahardjo et al., 2010b)

Figures 62 to 64 show that the groundwater table of the residual soil slopes from JF was generally deeper than that from BTG during dry and wet periods whereas the position of groundwater table of the residual soil slopes from OA was similar to that from BTG. The groundwater table of the residual soil slope from JF indicated a larger difference between dry and wet periods as compared to that from BTG. This condition may be attributed to the large variability of soil types in the residual soil from JF that was derived from the sedimentary rocks of different types from conglomerate to shale, siltstone and sandstone. On the other hand, the residual soil from BTG was derived mainly from the same granitic rock of Bukit Timah formation.

Numerical Modelling
Case Study
- Rainfall Intensity

The rainfall patterns in Singapore for JF, BTG and OA were represented by the rainfall at residual soil slopes from Slope 1-3, Slope 1-7 and Slope 3-7, respectively (Figure 65).
**Soil Properties**

The index soil properties and grain size distributions of residual soils at Slope 1-3, Slope 1-7 and Slope 3-7 are given in Figures 23 to 25. Soil-water characteristic curves (SWCC) for the soils are presented in Figure 26. Permeability functions for the residual soils were estimated using the previously mentioned statistical method and are presented in Figure 27.

The shear strength parameters for an unsaturated soil such as effective cohesion $c'$, effective angle of internal friction $\phi'$ and $\phi^b$ angle were determined from consolidated drained triaxial tests as shown in Figures 66 to 68. In the numerical analysis, the low retaining wall at the toe of Slope 1-3 was assumed to have a unit weight, $\gamma = 24 \text{ kN/m}^3$ and a low permeability, $k_s = 10^{-11} \text{ m/s}$. The retaining wall was assumed as a soil layer with an effective cohesion, $c' = 5 \text{ kPa}$ and an effective internal friction angle, $\phi' = 40^\circ$. 

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**Figure 65:** Rainfall and infiltration rate on (a) 26 September 2008 used in the analyses of residual soil slope 1-3 (b) 11 March 2008 used in the analyses of residual soil slope 1-7, and (c) 17 July 2008 used in the analyses of residual soil slope 3-7
- **Seepage Analysis**

  The software Seep/W (Geo-Slope International Pte. Ltd., 2007a) was used to conduct the seepage analysis. The finite element mesh used and the boundary conditions for the seepage analysis are presented in Figures 66 to 68 for the residual soil slope at Slope 1-3, Slope 1-7 and Slope 3-7, respectively. The left and right edges were located at a distance of \( 3H_s \) (\( H_s \) = slope height) from the crest and the toe, respectively, to avoid any influence of the boundary conditions on the seepage process within the slope area (Rahardjo et al., 2013a; 2012c). Transient seepage analysis was conducted by applying a small uniform infiltration to the surface of the slope for a long duration to represent the net flux of water on the site in order to obtain the initial profile of pore water pressure.

  Another transient seepage analysis was subsequently carried out for the actual rainfall. The flux boundary condition was assigned to the ground surface by considering the potential seepage face review. It would allow infiltration to enter at a rate depending on soil permeability and hydraulic gradient and the remaining amount of rainwater to runoff. No ponding condition was allowed in the seepage analyses. It means that the software will check that every node at the ground surface has zero or negative hydraulic head.

  The side boundaries below the groundwater table were specified as constant total head boundaries and the side boundaries above the groundwater table were specified as no flow zone or nodal flux, \( Q \), equal to zero. The applied total head corresponded to the hydrostatic condition. Tensiometer measurements in Singapore have been found to have a maximum negative limit of 75 kPa negative pore-water pressure (Rahardjo, 2000). Therefore, a hydrostatic pore-water pressure distribution with a limiting negative pore-water pressure of 75 kPa was set as an initial condition for the slope model in this project.

![Figure 66: Finite element mesh and boundary conditions of residual soil slope at Slope 1-3](image-url)
Slope Stability Analysis

Rainwater infiltrated into the unsaturated zone of the soil and increased the pore-water pressure. When the rainwater percolated downwards, the matric suction at deeper depths also decreased, causing a reduction in shear strength and the factor of safety of the slope. As the infiltrating rainwater reduced over time than the rainwater percolating downward into the slope, pore-water pressures above the wetting front began to decrease again, shear strength of the soil increased and consequently the factor of safety of the slope started to increase.

The program Slope/W (Geo-Slope International Pte Ltd., 2007b) was used to perform the limit equilibrium slope stability analysis. Bishop’s simplified method was selected in the analysis due to its capability in calculating factor of safety with an accuracy close to the more rigorous methods (Ching and Fredlund, 1984; Fredlund and Krahn, 1977). The slope stability analyses were carried out by importing the obtained pore-water pressure distributions from the seepage...
analyses. The time dependent pore-water pressure distributions were used to calculate the factor of safety with time.

- **Changes of $u_w$ with Time**

Comparison of pore-water pressure profiles obtained from the numerical analyses and field measurements at the middle of the residual soil slope from Slope 1-3, Slope 1-7 and Slope 3-7 are presented in Figure 69. It can be seen that the numerical analyses could simulate the process in the field with reasonably good agreement. Negative pore-water pressures approached zero near the ground surface due to rainfall.

![Figure 69: Pore-water pressure profiles obtained from numerical analyses versus field data of the residual soil slope at (a) Slope 1-3, (b) Slope 1-7, and (c) Slope 3-7 from the beginning of rainfall until the end of rainfall](image)

- **Variation of Factor of Safety**

Figure 70 shows the factor of safety with time from the slope stability analyses using Slope/W. The initial factor of safety of the residual soil slope at Slope 1-3 was 1.37 which decreased gradually to a value of 1.33 when the rainfall stopped (i.e. elapsed time of 5 hours) and continued to decrease to a minimum
value of 1.28 at 25 h after the rainfall stopped (i.e. elapsed time of 30 hours). The minimum factor of safety for the Slope 1-3 did not coincide with the time when the rainfall stopped, but occurred about 25 h after the rainfall stopped. The lower permeability of the residual soil slope from the JF resulted in a slower rainwater infiltration which delayed the occurrence of the minimum factor of safety to sometime later after the rainfall had stopped.

A steeper gradient of factor of safety reduction in Slope 1-7 and Slope 3-7 during rainfall illustrates the important role of permeability of the soil. The high permeability of the soil allowed rainwater to infiltrate and percolate downward quickly to deeper depths. As a result, a rapid increase in the pore-water pressure would develop in the slope. As a consequence, the factor of safety of Slope 1-7 decreased rapidly from initial value of 1.71 until it reached a minimum value of 1.57 when the rainfall stopped (i.e., at elapsed time of 6 hours). The factor of safety of Slope 3-7 decreased from 2.25 to the minimum value of 1.68 when the rainfall stopped (i.e., at elapsed time of 5 hours).

The deeper initial water table location in Slope 1-7 and Slope 3-7 might contribute to the higher initial factor of safety of the slope as compared to that of Slope 1-3. On the other hand, the factor of safety variation during rainfall is controlled by the applied rainfall characteristics (including rainfall pattern, intensity and duration) and the soil properties of the slope, particularly the coefficient of permeability. Figure 70 also shows that the recovery rate of factor of safety of the Slope 1-7 and Slope 3-7 after rainfall was faster than that of the
Slope 1-3 due to the higher permeability of the residual soil from the Bukit Timah Granite.

**Variation of Factor of Safety Based on Maximum Rainfall**

Two-dimensional seepage and slope stability analyses were also performed for 7 slopes at Jurong Formation, 7 slopes at Bukit Timah Granite and 3 slopes at Old Alluvium using finite element software, Seep/W and Slope/W. Based on the Code of Practice of Power and Utilities Board, Singapore (PUB, 2000) for drainage system design, the maximum total amount of rainfall in a day is 533.2 mm. Therefore, 22 mm/h rainfall intensity for 24 hour duration was applied in seepage analyses of the investigated residual soils to obtain the variation of factor of safety for residual soil slopes in Singapore under the worst condition. The initial position of groundwater table for each slope was drawn based on the average position of the actual groundwater table within each slope in one year.

Bishop’s simplified method was selected in the analysis due to its capability in calculating factor of safety with an accuracy close to the more rigorous methods (Ching and Fredlund, 1984; Fredlund and Krahn, 1977). The analyses incorporated unit weight of soil as well as the saturated and unsaturated shear strength parameters, $c'$, $\phi'$ and $\phi^b$ of the soil. Figures 71 to 73 show the variation of factor of safety with respect to time for residual soil slope from JF, BTG and OA.

![Figure 71: Factor of safety variations of residual soil slope from sedimentary Jurong Formation during and after 24 hours of rainfall](image-url)
Figure 72: Factor of safety variations of residual soil slope from Bukit Timah Granite during and after 24 hours rainfall

It was observed in Figure 71 that almost all factors of safety of residual soil slopes from JF decreased gradually to a critical (lowest) factor of safety during rainfall. Only the factor of safety for the residual soil slope at Slope 1-3 decreased significantly indicating a larger pore size of soil particle within the soil layer at Slope 1-3 as compared with other residual soil slopes from JF. Figure 71 also shows that the lowest factor of safety for 5 residual soil slopes from JF was observed several hours after rain stopped. The characteristics of factor of safety variations of the residual soil slope from JF during rainfall can be used to explain many incidences of slope failures that occurred sometime after rainfall had ceased (i.e., delayed slope failures). Figure 71 indicates that the slope preventive measures need to be installed within residual soil slope at Slope 2-10 since the factor of safety for this slope was low.

Figure 73: Factor of safety variations of residual soil slope from Old Alluvium during and after 24 hours rainfall

Figure 72 shows that the factor of safety for residual soil slope from BTG decreased significantly until it reached critical (lowest) value at the end of rainfall. This occurred due to the fact that water percolated down very fast through the soil layer until deeper depths, causing a rapid increase in the pore-
water pressures in the slope. The results of stability analyses demonstrated that rainwater infiltrated the residual soil slope from BTG at a faster rate than the infiltration rate through the residual soil slope from JF, resulting in the rapid decrease of factor of safety in the residual soil slope from BTG (Figures 71 and 72). The differing rates of infiltration could be attributed to the fact that the permeability of the residual soil from BTG was higher than that from JF. Figures 71 and 72 also show that the recovery of factor of safety of the residual soil slope from BTG after rainfall was faster than that of the residual soil slope from JF due to the higher permeability of the residual soil from BTG that allowed the infiltrating water to be drained quickly.

Figure 73 shows that the factor of safety variations for residual soil slope from OA followed the same pattern as those for residual soil from BTG. The factor of safety for residual soil slope from OA decreased significantly until it reached critical (i.e., lowest) value after the end of the rainfall. This phenomenon also occurred due to the fact that permeability of residual soil from OA was high and water rapidly percolated down through the soil layer until deeper depths, causing a rapid increase in the pore-water pressures in the slope.

**Variation of Groundwater Table Position**

Figure 74 shows a variation in factor of safety for the slopes from JF and BTG for different locations of ground water table with a slope height of 15 m and slope angle of 27°. The higher the groundwater tables from the surfaces of the slopes from JF and BTG or the thinner the unsaturated zone, the lower the factor of safety would be.

![Figure 74: Factor of safety for the (a) sedimentary Jurong Formation and (b) granitic Bukit Timah for different location of ground water table with 15 m of slope height and 27° of slope angle](image)

Variations of factor safety due to initial groundwater table positions at the residual soil slopes from JF and BTG are given in Figure 75a and 75b,
respectively. Figure 75a shows that the factor of safety decreased rapidly during the driest period due to the initially high matric suction of the soil above the groundwater table before rainfall. The decrease in factor of safety was slower in the wettest period as compared to other periods.

The factor of safety \((F_s)\) was 2.5 for the slope from JF in the driest period. The factor of safety decreased significantly to a minimum factor of safety of 1.8 one hour after the rainfall stopped. After which, the factor of safety recovered slowly. Figure 75a also shows that the factor of safety \((F_s = 2.0)\) of the slope from JF at the average condition decreased rapidly and reached a minimum factor of safety of 1.6 one hour after the rainfall stopped. Subsequently, it recovered at the same rate as that for the driest period. The factor of safety \((F_s = 1.56)\) of the slope from JF at the wettest period decreased slowly and it reached a minimum value of 1.4, 12 hours after the rainfall stopped. The time delay could be attributed to the effects of SWCC and permeability function on the residual soil from JF. The residual soil slope from JF had a lower water-entry value and gentler permeability function, resulting in a slower infiltration rate of rainwater as compared to that in the BTG slope. Although rainfall already stopped, the rainwater continued to percolate down into greater depths. As a result, the most critical slip surface might be observed several hours after the rainfall stopped.

Figure 75b shows that the factor of safety for the slope from BTG reached a minimum value when the rainfall stopped regardless of the position of ground water table. The factor of safety for the slope from BTG in the driest period and at the average condition decreased drastically and reached a minimum factor of safety of 1.7. After the rainfall ceased, the factor of safety for the BTG slope at the driest period and at the average condition increased rapidly at the same recovery rate. The factor of safety for the slope from BTG in the wettest period dropped gently from a factor of safety 1.7 to 1.45 and it increased slowly after the rainfall stopped.

![Figure 75: Variation of factor of safety for the slopes from (a) sedimentary Jurong Formation and (b) granitic Bukit Timah for different groundwater table position (slope height = 15 m and slope angle = 27°)](image-url)
Variation of Rainfall Intensity

This section presents the effect of rainfall intensity on the stability of the slopes from JF, BTG and OA in Singapore. All series in these parametric studies were performed using the average groundwater table positions of residual soil slopes in Singapore (Figures 63 to 65) as the initial locations of groundwater table. Four different rainfall intensities, 9, 22, 36 and 80 mm/h were used. The rainfall intensity of 9 mm/h corresponds to a return period of 100 years based on the intensity-duration-frequency (IDF) curve for Singapore (PUB, 2000). Paulhus (1965) observed that the greatest rainfall intensity in the world was 80 mm/h for 24 hours. Therefore, this rainfall intensity was also adopted in the numerical analyses. All numerical analyses were conducted for a period of 48 hours with the application of rainfall during the first 24 hours.

The effects of different rainfall intensities on the stability of the slopes from JF and BTG are shown in Figures 76a and 76b, respectively. The factor of safety for the slope from BTG with a rainfall higher than 22 mm/h dropped drastically during the rainfall period and recovered rapidly after the rainfall stopped. The magnitude and rate of decrease in factor of safety were directly proportional to the magnitude of rainfall intensity. However, the minimum factors of safety of the slope from BTG with 22, 36, and 80 mm/h of rainfall were slightly different. This behaviour indicated that the slope had reached its threshold rainfall intensity at 22 mm/h. On the other hand, a rainfall of 9 mm/h had no significant effect on the factor of safety for the slope from BTG. Since slope from BTG had a low permeability at a high suction, the deep groundwater table of the slope from BTG created a high matric suction in the slope from BTG before rainfall and hence the permeability was low. The application of 9 mm/h of rainfall for 24 hours was not high enough to change the matric suction of the soil from BTG significantly. Therefore, the rainwater had difficulties infiltrating into the slope and the factor of safety remained constant during 24 hours of rainfall.

Rainwater infiltrated into the slope slowly during 12 hours of rainfall causing the factor of safety to drop gradually in the slope from JF (Figure 75a). Similar to the soil from BTG, the permeability of the soil from JF was low due to the initially high matric suction of the slope from JF. After 12 hours of rainfall, the matric suction of the soil from JF had already decreased significantly and permeability of the soil also increased. Therefore, the factor of safety started to decrease drastically and recovered gradually after reaching the minimum value of factor of safety. The minimum factor of safety was reached several hours after the rainfall stopped regardless the intensity of rainfall. The slope from JF with 9, 22 and 36 mm/h rainfall reached the minimum factor of safety, one hour after the
rainfall stopped. Meanwhile, the slope from JF with 80 mm/h of rainfall reached the minimum factor of safety of 1.64, three hours after the rainfall stopped (Figure 75a). The delay in reaching the minimum factor of safety occurred due to the low saturated permeability of the soil from JF. After the rainfall ended, it took some time for the rainwater to reach the critical point depending on the infiltration rate of the rainwater. The minimum factors of safety of the JF slope with an applied rainfall of 22, 36 and 80 mm/h were slightly different. It indicated that the rainfall intensity of 22 mm/h can be used as a threshold value for seepage analyses of residual soil slopes in Singapore.

Figure 76b shows that the factor of safety for the slope from BTG remained constant during a low intensity of rainfall (9 mm/h) whereas the factor of safety for the slope from JF (Figure 75a) decreased quite significantly for the same rainfall intensity. It can be attributed to the fact that the permeability of the soil from BTG was lower at high suctions than that of the soil from JF. As a result, rainwater infiltrated in the slope from BTG at a slower rate than the infiltration rate in the slope from JF, causing the factor of safety for the slope from BTG to remain essentially constant while the factor of safety for the slope from JF to decrease significantly.

Figure 76: Variation of factor of safety for the slopes from (a) sedimentary Jurong Formation and (b) granitic Bukit Timah for different rainfall intensities (slope height = 15 m and slope angle = 27º) (Rahadjo et al., 2008b)

**Variation of Factor of Safety due to Variation of Soil Properties**

The hydraulic properties of soil (SWCC, saturated permeability and unsaturated permeability function) are the primary data required for transient seepage analyses to assess the stability of unsaturated soil slope. It is important to fully understand the effect of each property on the stability of soil slopes subjected to different rainfall intensities as discussed in this section. Typical slope geometry and boundary conditions used in this study shown in Figure 77. The values selected for slope angle and slope height were based on typical slope
geometry in Singapore (Toll et al., 1999). The slope angle, α, was selected to be \(30^\circ\) and slope height, \(H_s\), was selected to be 15 m.

The initial water table was selected to be a straight line with \(5^\circ\) inclination with respect to the horizon and 2 m deep based on a typical value for depth of water table in Singapore. Figure 77 shows the boundary conditions used for the transient seepage analysis. Boundary flux, \(q\), equal to the desired rainfall intensity was applied to the surface of the slope. Evaporation and evapotranspiration were not considered. The boundary flux, \(q\), applied was reviewed by elevation to simulate rainfall loading without ponding. The nodal flux, \(Q\), equal to zero was applied along the sides of the slope above the water table line and along the bottom of the slope to simulate no flow zone. The boundary condition along the sides of the slope below the water table line was taken to be total head boundary with values equal to the total head, \(h_w\), along the sides.

![Figure 77: Slope geometry and boundary conditions for the homogeneous soil slopes](image)

The shear strength properties of the soil used in this study were chosen based on typical shear strength properties of soil in Singapore (Rahardjo et al., 2007f). The effective cohesion, \(c' = 10\) kPa, effective angle of internal friction, \(\phi' = 26^\circ\), rate of increase in shear strength caused by matric suction, \(\phi^b = 26^\circ\), and unit weight of the soil, \(\gamma = 20\) kN/m\(^3\), were used in slope stability analyses. The shear strength parameters of the soil were kept constant in all cases to ensure that the changes in the stability of the slope were only due to pore-water pressures (or matric suction) changes in the soil.

The hydraulic properties of soil could be quantified in terms of the fitting parameters of the SWCC equation of Fredlund and Xing (1994) with a correction factor equal to 1 (Leong and Rahardjo, 1997) (Equation 3) and the associated permeability function computed from the SWCC and the saturated coefficient of permeability, \(k_s\). As a result, the fitting parameters \(a, m, n\) and saturated coefficient of permeability, \(k_s\), are the variable parameters in this parametric study. Four sets of parametric studies were conducted to investigate the role of all these
parameters. Soil types were selected to represent the residual soil of tropical regions and categorized into good and poor drainage soils based on the saturated coefficient of permeability, \( k_s \).

\[
\theta_w = C(\psi) \frac{\theta_s}{\ln \left( e + \left( \frac{u_a - u_w}{a} \right)^n \right)^m}
\]

where: \( \theta_w \) = calculated volumetric water content; \( \theta_s \) = saturated volumetric water content; \( e \) = natural number (2.71828); \( a, n, m \) = fitting parameters; \( C(\psi) \) = correction factor = 1.

Four combinations of parametric study were performed for each category of soil. For every combination, one variable parameter was considered to have three different values while other parameters remained constant. In set A, parameter “\( a \)” was set to be 5, 10 and 20 for good drainage and 100, 500 and 1000 for poor drainage soil. In set B, parameter “\( m \)” was set to be 0.5, 1 and 2 for good and poor drainage soil. In set C, parameter “\( n \)” was set to be 1, 2 and 3 for good and poor drainage soil. In set D, saturated coefficient of permeability was set to be \( 10^{-4} \), \( 5 \times 10^{-5} \) and \( 10^{-5} \) m/s for good drainage soil and \( 10^{-6} \), \( 5 \times 10^{-7} \) and \( 10^{-7} \) m/s poor drainage soil. The values of the parameters used in the study were selected from experimental data.

The soils were examined under various rainfall intensities of 24 hours duration to study the effect of short duration rainfall on the stability of slopes. Rainfall intensities applied to the good drainage soil slopes are 0.01 \( k_s \), 0.05 \( k_s \), 0.25 \( k_s \), 0.5 \( k_s \) and 1 \( k_s \) and rainfall intensities applied to the poor drainage soil slopes are 0.01 \( k_s \), 0.05 \( k_s \), 0.1 \( k_s \), 0.25 \( k_s \), 0.5 \( k_s \), 1 \( k_s \), 2.5 \( k_s \) and 5 \( k_s \).

### Effect of Parameter “\( a \)”

Figures 78a and b presents the variation of normalized factor of safety, \( F_{sn} \), versus time for good and poor drainage soil slopes with different values of parameter “\( a \)”, respectively. Applying a rainfall intensity of 0.25 \( k_s \) for duration of 24 hours resulted in a variation in \( F_{sn} \). Figure 78 shows that the higher parameter “\( a \)” in both good and poor drainage soils, the faster the rate of decrease in the normalized factor of safety, \( F_s \), versus time. The soil with a higher value of parameter “\( a \)” has a faster rate of recovery in \( F_{sn} \), after rainfall stopped and lower minimum normalized factor of safety, \( F_{sn(min)} \). Similar patterns were observed for other rainfall intensities.
Figure 79 presents the minimum normalized factor of safety versus rainfall intensity for different parameters “a” for good and poor drainage soils. Regardless of soil type, the higher the value of parameter “a”, the lower the minimum normalized factor of safety. For all rainfall intensities, the difference between the minimum normalized factors of safety for all the good drainage soil type was relatively small. It means that the variation of the parameter “a” for the good drainage soil has insignificant effect on the stability of the slope. On the other hand, the difference between the minimum normalized factors of safety for the poor drainage soil was significant. Therefore, variation in the parameter “a” had a significant effect on slope stability of the poor drainage soil.

For good drainage soil, Figure 79a describes that the lowest value of minimum normalized factor of safety corresponded to a rainfall intensity lower than the saturated coefficient of permeability ($k_s = 10^{-4}$ m/s or 360 mm/h). However, for poor drainage soil, the minimum normalized factor of safety corresponded to a rainfall intensity much higher than the saturated coefficient of permeability ($k_s = 10^{-6}$ m/s or 3.6 mm/h) shown in Figure 79b.
With respect to the SWCC, a higher value of parameter “a” means a higher volumetric water content at a given matric suction. In unsaturated soil, water flows through the pore spaces that are re-filled with water. Therefore, in soil with a higher volumetric water content, the movement of water is faster (Fredlund and Rahardjo, 1993). With respect to the permeability function, $k_w$, the soil with a higher value of parameter “a” has a higher permeability function and therefore, a higher infiltration rate of rainwater into the soil. As a result, the rate of reduction in factor of safety is faster for a soil with a higher value of parameter “a”.

- **Effect of Parameter “m”**

Figure 80 shows the variation in normalized factor of safety, $F_{sn}$, versus time for different values of parameter “m” for both good and poor drainage soil slopes under 24-hour rainfall at an intensity of 0.25$k_s$. Regardless of soil types, the rate of decrease in the normalized factor of safety, $F_{sn}$, versus time was faster for soil with a lower value of parameter “m” = 0.5. The rate of recovery in $F_{sn}$ after rainfall had stopped was also faster for a lower value of parameter “m”. Also the minimum normalized factor of safety, $F_{sn(min)}$, was lower for soil with a lower value of parameter “m”. The same pattern was observed for other rainfall intensities.

![Figure 80: Comparison of normalized factor of safety, $F_{sn}$, versus time for different values of constant parameter “m”: (a) good drainage soil rainfall intensity, $I_r = 90$ mm/h = 0.25$k_s$, (b) poor drainage soil rainfall intensity, $I_r = 0.9$ mm/h = 0.25$k_s$ (after Rahimi et al., 2010)](image)

Figure 81 presents the minimum normalized factor of safety versus rainfall intensity for different parameters “m” for good and poor drainage soils. Both figures show that regardless of soil type, the lower the value of parameter “m”, the lower the minimum normalized factor of safety. Figure 81a shows that the reduction in the minimum normalized factor of safety was large for all good drainage soils. However, the difference in the amount of reduction was small. It
appears that the variation in the parameter “m” did not have a significant effect on the stability of the good drainage soil slopes. Figure 81b shows that the reduction in the minimum normalized factor of safety was not large for poor drainage soil. However, the difference in the amount of reduction was significant. In other words, the variation of the parameter “m” had a significant effect on the stability of poor drainage soil slopes.

Figure 81: Minimum normalized factor of safety versus rainfall intensity for variation of the parameter “m”: (a) good drainage soil, (b) poor drainage soil (after Rahimi et al., 2010)

Parameter “m” in SWCC is related to the residual water content, \( \theta_r \), of the soil. As the value of the parameter “m” decreased, the residual water content of the soil increased. In addition, the volumetric water contents at all matric suction values were higher for the soil with a lower value of parameter “m”. For a soil with higher water content, the permeability is higher. Therefore, the movement of water through soil with a lower value of parameter “m” is faster and rainwater infiltrates faster into the soil. As a result, the soil with a lower value of parameter “m” experiences a higher rate of reduction and recovery in the factor of safety and had the lowest minimum factor of safety.

- **Effect of Parameter “n”**

Figure 82 presents the variation of normalized factor of safety, \( F_{sn} \), versus time for different values of parameter “n” for good and poor drainage soil slopes under 24-hour rainfall at an intensity of 0.25\( k_s \). The difference in the rate of decrease and recovery in the normalized factor of safety, \( F_{sn} \), versus time was not significant for good drainage soils as shown in Figure 82a. Also, the minimum normalized factor of safety, \( F_{sn(min)} \), was more or less the same for different values of parameter “n”. However, as shown in Figure 81b, the rate of decrease in the normalized factor of safety, \( F_{sn} \), versus time was faster for the poor drainage soil with a higher value of parameter “n”. The rate of recovery in \( F_{sn} \) after rainfall had stopped was faster for a higher value of parameter “n”. In addition, the minimum
normalized factor of safety, $F_{sn(min)}$, was lower for the soil with a higher value of the parameter “$n$”. The same pattern was observed for other rainfall intensities.

Figure 82: Comparison of normalized factor of safety, $F_{sn}$, versus time for different values of constant parameter “$n$”: (a) good drainage soil rainfall intensity, $I_r = 90$ mm/h = 0.25$k_u$, (b) poor drainage soil rainfall intensity, $I_r = 0.9$ mm/h = 0.25$k_u$ (after Rahimi et al., 2010)

Figure 83 shows minimum normalized factor of safety versus rainfall intensity for different values of parameters “$n$” for good and poor drainage soils. Figure 83a shows that the reduction in the normalized factor of safety was large for all good drainage soils. However, the amount of reduction was the same for all values of parameter “$n$”. In fact, variation of the parameter “$n$” did not have any effect on the stability of the good drainage soil slopes. Figure 83b shows that the difference in the amount of reduction in the minimum normalized factor of safety was significant for poor drainage soils. In fact, variation of the parameter “$n$” had a significant effect on the stability of poor drainage soil slopes.

Figure 83: Minimum normalized factor of safety versus rainfall intensity for variation of the parameter “$n$”: (a) good drainage soil, (b) poor drainage soil (after Rahimi et al., 2010)

The parameter “$n$” affects the slope of SWCC. For a higher value of parameter “$n$”, the slope of SWCC was steeper. The volumetric water content of soil with a higher value of parameter “$n$” was higher for matric suctions lower than the air-entry value and was lower for matric suctions higher than air-entry
value. Therefore, the response of a soil slope under rainfall loading (i.e. variation of the factor of safety) is controlled by the initial matric suction of the soil and whether its magnitude is lower or higher than the air-entry value of the soil. For the poor drainage soils, the initial matric suction of the soil was lower than the air-entry value of the soil. Therefore, the infiltration process was faster in a soil with a higher value of parameter \( n \), resulting in a higher rate of reduction and recovery in factor of safety and the lowest value of the minimum normalized factor of safety.

The summary of the effects of hydraulic properties of soil on slope stability under rainfall conditions is presented in Table 5.

<table>
<thead>
<tr>
<th>Variable parameters</th>
<th>Rate of decrease and recovery in ( F_{sn} )</th>
<th>( F_{sn(min)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good drainage ( (k_s \geq 10^{-4} \text{ m/s}) )</td>
<td>Poor drainage ( (k_s \leq 10^{-6} \text{ m/s}) )</td>
<td>Good drainage ( (k_s \geq 10^{-4} \text{ m/s}) )</td>
</tr>
<tr>
<td>Increase in “( a )”</td>
<td>increase</td>
<td>Increase</td>
</tr>
<tr>
<td>Increase in “( m )”</td>
<td>decrease</td>
<td>Decrease</td>
</tr>
<tr>
<td>Increase in “( n )”</td>
<td>not sensitive</td>
<td>Increase</td>
</tr>
</tbody>
</table>

**Effect of Different Antecedent Rainfall Patterns**

The effect of antecedent rainfall pattern on the stability of the slope was discussed in this parametric study. Actual rainfall data from many regions of Singapore were analysed to identify repeatable rainfall patterns. The identified rainfall patterns were then applied to two different soil types which represent high and low conductivity residual soils of Singapore. A major rainfall event based on actual rainfall data was then applied to slopes after the application of the antecedent rainfall. Seepage and slope stability analyses were performed in order to study the stability of the slope subjected to various antecedent rainfall patterns. Transient seepage analysis was performed to compute the pore-water pressure. Subsequently, the computed pore-water pressures were used to calculate the factor of safety for the slope during rainfall.

The slope geometry, boundary condition and shear strength properties used in this part of the study were the same with the one used in the previous section. The different patterns of antecedent rainfall could be classified into three different categories as shown in Figure 84.
Figure 84: Actual and idealized rainfall patterns for rainfall data of December 2006: (a) increasing intensity towards the end of rainfall, (b) maximum intensity at the middle of rainfall and (c) decreasing intensity towards the end of rainfall (after Rahimi et al., 2011)

Figure 84a shows that rainfall began at low intensity and gradually increased to the end. Figure 84b shows that rainfall started at low intensity in the beginning of rainfall, gradually increased at the middle and then decreased gradually at the end of rainfall duration. Figure 84c shows that rainfall started at a high intensity in the beginning and then decreased gradually at the end of rainfall duration. For the three recognized rainfall patterns, the maximum continuous 5-day rainfall, 450 mm, was distributed based on the idealized rainfall patterns and multiplied by the idealized rainfall percentage for each time interval. Total rainfall in each time interval was divided by 8 hours to obtain rainfall intensity.

Figure 85 shows that the rainfall patterns affected the rate of reduction in $F_s$ and corresponding to $F_{s(min)}$ for both high conductivity (HC) and low conductivity (LC) soil slopes. The $F_s$ of HC soil slopes decreased by 13% from its initial value, while the $F_s$ of LC soil slopes decreased by 45% of its initial value.

It is shown that for HC slopes the rate of decrease in factor of safety was faster for the advanced pattern, followed by the normal and delayed patterns. The minimum factor of safety, $F_{s(min)}$, equal to 1.53, 1.51 and 1.48 happened at 56, 88 and 120 hour for the advanced, normal and delayed patterns, respectively. In other words, the advanced rainfall pattern resulted in the earliest time and highest value
for the $F_{\text{S(min)}}$ to occur as compared to the normal and delayed rainfall patterns. The difference in $F_{\text{S(min)}}$ for all the rainfall patterns was relatively small. The rate of recovery of factor of safety was fastest for the delayed pattern followed by the normal and advanced patterns.

Figure 85 also shows the rate of decrease in factor of safety for LC slopes was fastest for the advanced pattern followed by normal and delayed patterns. The minimum factor of safety, $F_{\text{S(min)}}$, equal to 1.001, 1.004 and 1.083 happened at 96, 112 and 120 hour for the advanced, normal and delayed patterns, respectively. The advanced rainfall pattern resulted in the earliest time and lowest value for the $F_{\text{S(min)}}$ to occur as compared to the normal and delayed rainfall patterns. The difference in $F_{\text{S(min)}}$ for all the rainfall patterns was also relatively small. The rate of recovery of factor of safety was slower for the delayed pattern compared to the normal and advanced patterns.

![Figure 85: Normalized factor of safety versus time for various rainfall patterns (after Rahimi et al., 2011)](image)

**Preventive Measures**

Negative pore-water pressure or matric suction in the unsaturated zone between the groundwater table and the slope surface contributes significantly to the shear strength of residual soil and consequently to the stability of the residual soil slope. However, matric suction can decrease due to infiltration of rain water from the slope surface or due to the rising of water table or both during rainfalls. Therefore, matric suction or the unsaturated soil zone has to be maintained using some preventive measures as illustrated in Figure 86. Horizontal drains are recognized as the most economical method available for lowering and maintaining groundwater level. The role of horizontal drains in maintaining the stability of a residual soil slope during heavy rainfalls was studied through field measurements of matric suction and numerical analyses. The protective cover
system is required to minimize rainwater infiltration into slope and to maintain the negative pore-water pressure in the unsaturated zone. The effectiveness of Capillary barrier system (CBS) and vegetative cover as a protective cover system to minimize rainwater infiltration into slope was also evaluated in this study.

Figure 86: Protective cover system and horizontal drain for maintaining soil suction during rainfall (modified from Rahardjo et al., 2003)

**Horizontal Drains**

The effectiveness of a horizontal drainage system is governed by several factors, such as drain type, location, number, length and spacing (Kenney et al., 1977; Nonveiller, 1981; Lau & Kenney, 1984; Nakamura, 1988; Martin et al., 1994; Prellwitz, 1978; Rahardjo et al., 2009), without disregarding the importance of soil properties and slope geometry as controlling parameters. In practice, regular maintenance and frequent replacement of horizontal drains are needed to maintain the performance of horizontal drain with time. Rahardjo et al. (2003) concluded that it is important to install horizontal drains near the toe of the slope to lower the groundwater table and consequently to lower the pore-water pressures.

The sedimentary Jurong Formation residual soil slope at Slope 1-10 was selected for the installation of horizontal drains. The geometry and installation layout is shown in Figure 87. The slope surface was well grassed and there were some trees on the right and left hand sides of the observed slope. There was no previous history of slope failure in this area including during the period of monitoring. Nevertheless, significant high groundwater levels during heavy rainfall were detected through monitoring of piezometers installed at the site.
Therefore, rectification work using horizontal drains was implemented to increase the factor of safety of the slope due to its proximity to a residential area.

The soil of the slope was classified as sandy silt based on Unified Soil Classification System (USCS), with a unit weight of 17.5 kN/m³, an effective cohesion of 5 kN/m², an effective friction angle of 26° and an angle indicating the rate of increase in shear strength relative to the matric suction, $\phi^b$ angle, of 15°. The water content, specific gravity, liquid limit and plastic limit of the soil were 38-40%, 2.65, 61% and 38.8%, respectively.

Figures 88 shows the pore-water pressure profiles measured by tensiometers near the crest, at the middle and near the toe of the slope on 21 August 2008. It can be seen that the pore-water pressures near the crest and the middle of the slope can be as low as -60 kPa and -50 kPa near the ground surface, respectively, during the dry period as measured on 21 August 2008 and tend to be more negative at deeper depths. Maximum changes in pore-water pressure occurred near the ground surface and the magnitude of the changes decreased with depth. Pore-water pressures near the ground surface were the first to be affected by heavy rainfall, followed by those at greater depths. Extended rainfalls caused a significant change in negative pore-water pressure towards positive pore-water pressure as observed at each depth of tensiometer.

Figure 87: Cross-section of the instrumented slope
Finite element software (Seep/W) was used to simulate the unsaturated groundwater flow and to determine the pore-water pressure variation with time. Meanwhile, the slope stability analysis was conducted using Slope/W. Comparison of pore-water pressure profiles obtained from the numerical analyses and field measurements near the crest, middle and the toe of the slope are presented in Figures 89. It can be seen that the pore-water pressure profiles obtained from the numerical analyses near the crest and the middle of the slope were very close to the data measured in the field.

Near the toe of the slope (Figure 89c) the groundwater table in the field was initially lower than the horizontal drain position at a distance of $0.4L$ ($L =$ horizontal drain length) from the toe of the slope, while beyond this point the groundwater table was higher than the position of the horizontal drain. In the numerical analyses, the initial profile of pore-water pressure was obtained by applying a small uniform infiltration rate to the slope model for a long duration.
prior to applying the actual rainfall. As a result of establishing the initial condition in the numerical analyses, the groundwater table dropped to the drain level in the upper part of the slope, but rose near the toe of the slope. Therefore, it appears that the pore-water pressures near the toe of the slope increased quickly in the numerical analyses, but this was not the case in the field where the actual pore-water pressures remained negative as reflected by the tensiometer measurements.

![Figure 89: Comparison of pore-water pressure profiles obtained from numerical analyses and pore-water pressure data measured in the field (a) near the crest, (b) at the middle and (c) near the toe of the slope from the beginning of rainfall (t=0) until the end of rainfall (t=21 day) (Rahardjo et al., 2011b)](image_url)

By importing the pore-water pressure distribution resulted from Seep/W, factors of safety were computed for the slope model with and without horizontal drains in Slope/W using the Bishop’s simplified method of slices. The variations of factor of safety with respect to time during heavy rainfall are shown in Figure 90. At all times, the factor of safety of the slope with horizontal drains was higher than the factor of safety of the slope without horizontal drains, indicating that horizontal drains improved the stability of slope, especially during heavy rainfall events. In addition, for both cases with or without horizontal drains, the minimum factor of safety did not occur at the end of rainfall, but at a much later time due to the low permeability of Jurong Formation residual soil layer which caused water percolated downward slowly.
Capillary Barrier System

A capillary barrier is an earthen cover system designed as an unsaturated system using a fine-grained layer of soil overlying a coarse-grained layer of soil (Nicholson et al., 1989; Ross, 1990a, b; Stormont, 1996; and Rowlett & Barbour, 2000). The principle of the capillary barrier system is based on the difference in unsaturated hydraulic properties (soil-water characteristic curves and permeability functions) of each material (Rahardjo et al., 2007a). Under unsaturated conditions, the difference in permeability between the fine-grained layer and the coarse-grained layer limits the downward movement of water through capillary barrier effect. The infiltrated water is stored temporarily in the fine-grained layer (Rahardjo et al., 2007b) and then removed by lateral drainage through the slope, evaporation (Bruch, 1993) and evapotranspiration, minimizing percolation into the underlying layer. When percolation (breakthrough) takes place, the capillary barrier no longer impedes water from infiltrating into the slope (Rahardjo et al., 2009a).

In this study, two sites (i.e. Slope 1-4 and Slope 3-7) were selected for installation of capillary barrier system. The increase in stability during heavy rainfall by capillary barrier system was evaluated by field measurements of the pore-water pressures.

- **Capillary Barrier System at Slope 1-4**

The slope located at Slope 1-4 consisted of a residual soil from the Bukit Timah Granite and was protected by the capillary barrier system. The geometry of the slope is shown in Figure 91a. The groundwater table was located about 3.5 m below the ground surface of the middle of the slope.
Test results indicated that the specific gravity, liquid limit, and plastic limit range of the residual soil are 2.64–2.68, 53–66%, and 36–38%, respectively. The slope consisted of silty sand and elastic inorganic silt with moderate to high plasticity (Sand-Silt) with a unit weight of 13.52 kN/m³, an effective cohesion of 8 kPa, an effective friction angle of 33°, and a $\phi$ angle of 25°. The fine-grained material used in the first layer of the capillary barrier system was fine sand. The coarse-grained material used in the second layer of the capillary barrier system was granite chip. The fine-grained layer was compacted to relative density ($D_r$) between 70% and 90% or to the required dry density ($\rho_d$) of 1.56 Mg/m³. The coarse-grained layer was compacted to relative density ($D_r$) between 70% and 90% or to the required dry density ($\rho_d$) of 1.65 Mg/m³.

Figure 92 shows the detail of CBS construction at slope 1-4. Geocells were used to hold coarse- and fine-grained materials within CBS. Steel J-pins were used to secure the overlying geocells onto the ground. A layer of geofabric was laid on top of the coarse-grained layer to act as a separator between the coarse and fine-grained layers. Manual tamping was carried out to compact the coarse- and fine-grained layers. Once CBS was completed, jet-fill tensiometers were installed to monitor the variations of pore-water pressure during dry and wet periods (Figure 93).
Figure 92: Construction of CBS at slope 1-4 (Rahardjo, 2010a)

Figure 93: Jet-fill tensiometer for measurement of negative pore-water pressure within CBS at slope 1-4 (Rahardjo, 2010a)

Figure 94 shows variations of pore-water pressures with time of the original slope and the slope covered with CBS during dry and wet periods. Generally, the pore-water pressure at the crest of the slope with capillary barrier system increased gradually due to rainfall events but was still able to maintain the negative pore-water pressure, while the pore-water pressure at the crest of the original slope increased rapidly due to rainfall. The pore-water pressure at the depth of 2.0 m on the slope protected by the capillary barrier remained negative (from -70 to -5 kPa), while the pore-water pressure at the depth of 2 m in the slope without capillary barrier increased to a positive pressure of 20 kPa.
Based on the rainfall data, a rainfall event occurred at the location of the slope from 4 to 23 July 2008. The total rainfall was 242.6 mm for about 24 days and the maximum rainfall intensity was 45.6 mm/h on 7 July 2008. The pore-water pressure profiles near the crest of the slope with the capillary barrier system and of the original slope on 4 July 2008 were still highly negative prior to the rainfall events as presented in Figures 95a and 96a. The pore-water pressures increased during the rainfall events as reflected by the tensiometers measurements on 25 July 2008 as shown in Figures 95a and 96a. The measurement results indicated that the pore-water pressures in the original slope increased faster than the pore-water pressures in the slope with the capillary barrier system during rainfall.

Figures 95b and 96b show the drying processes in the slope with the capillary barrier system and the original slope after the rainfall event stopped (from 25 July 2008 onwards). The negative pore-water pressure started to develop again at the crest of the slope. In general, during the drying process, the pore-water pressures in the slope with the capillary barrier system moved towards higher negative values than those in the original slope. The wetting and drying processes show that the capillary barrier system was effective in minimizing the change in pore-water pressure during rainfall.
Capillary Barrier System at Slope 3-7

Slope 3-7 which had no prior experience of slope failure was selected for the construction of the CBS. The slope consisted of soil from the Old Alluvium. The slope had a height of 8.2 m, a length of 22.6 m, and a slope angle of 20°. The area of the slope covered with a CBS was about 40 m² in the middle section of the slope (Figure 97).
Comparison of pore-water pressure profiles between the slope with the capillary barrier system using recycled crushed concrete as the coarse-grained layer and the slope with the capillary barrier system with Secudrain as the coarse-grained layer are presented in Figures 98a and 98b, respectively. The modification in SWCC test set up was performed to obtain the SWCC data of recycled crushed concrete more accurately as explained in Rahardjo et al. (2011e). The data were obtained from both numerical analyses and field measurements during the rainfall event on 30 July 2010. Meanwhile, comparison of pore-water pressure profiles of the original slope obtained from the numerical analyses and field measurements during the rainfall event on 30 July 2010 are presented in Figure 95c.
Figure 98: Pore-water pressure profile during the rainfall event on 30 July 2010 for (a) the slope with the capillary barrier system with recycled crushed concrete as the coarse-grained layer, (b) the slope with the capillary barrier system with Secudrain as the coarse-grained layer, (c) original slope (Rahardjo et al., 2013b)

The numerical analyses showed a reasonably good agreement in the trend of the pore-water pressure profiles with those obtained from field measurements. The results of the seepage analyses and field measurements of the original slope point out that there was positive pore-water pressures built up to 6.8 kPa at an elevation of 111.6 m, indicating that there is an existence of a lower permeability soil layer \( (6.4 \times 10^{-7} \text{ m/s}) \) at the depth of 1.5 to 2.1 m from the slope surface. The monitoring results showed that the both capillary barrier systems with recycled crushed concrete or Secudrain as the coarse-grained layer the slope with the capillary barrier system were able to maintain negative pore-water pressures and effective in minimizing rainwater infiltration and maintaining stability of the slope (Rahardjo et al., 2012b). The presence of negative pore-water pressure contributed to the shear strength of the soil, causing the slope to be less vulnerable.
to failure. On the other hand, the pore-water pressure under the original slope was easily affected by rainwater infiltration.

Stability analyses of the original slope and the CBS with RCA as the coarse-grained layer and the CBS with Secudrain as the coarse-grained layer at Slope 3-7 were carried out by incorporating the pore-water pressure measured from 30 July 2010 to obtain the variation in factor of safety at Slope 3-7 using the Bishop’s simplified method of slices. Figure 99 presents the variations of factor of safety with respect to time during and after rainfall. The initial factor of safety of the original slope was 2.29 and decreased rapidly during rainfall until the minimum factor of safety of 1.85. The initial factor of safety of the capillary barrier system with RCA was 3.00 and decreased slightly to the minimum factor of safety of 2.97 due to the rainfall. Meanwhile, the initial factor of safety of the capillary barrier system with Secudrain was 2.42 and also decreased a little to the minimum factor of safety of 2.41 due to the rainfall. Higher negative pore-water pressures before rainfall in the slope with the capillary barrier system than those in the original slope contribute to the higher initial factor of safety of the slope with the capillary barrier system.

![Figure 99: Factor of safety variation during and after rainfall event on 30 July 2010 (Rahardjo et al., 2013b)](image)

**Vegetative Cover**

Green technology is an integrated design approach that combines vegetation and engineering design methods to mechanically reinforce slopes, control erosion, improve aesthetics of the environment, provide visual and noise barrier and to improve biological diversity (Woods 1938, VanDersal 1938, Marchent and Sherlock 1984, Greenway 1987, Coppin & Richards 1990, Menashe 1993, Meyers 1993, Gray & Sotir 1996, Norris et al. 2008, and Glendinning et al. 2009). In this project, the effect of Orange Jasmine and Vetiver
grass in increasing shear strength of soil and minimizing rainwater infiltration for maintaining stability of slope during rainfall was investigated on a soil slope in Singapore with its tropical climate.

Slope 3-7 with residual soil from Old Alluvium which had no prior record of slope failure was chosen for the construction of the green technology slope. A portion of the slope (50 m²) was covered by Orange Jasmine, another portion of the slope (50 m²) was covered by Vetiver grass, and another portion of the slope was the original slope with cow grass cover. The slope has a height of 8.2 m, a length of 22.6 m, and a slope angle of 20°. The schematic diagram of the slope is shown in Figure 100.

Slope stability analyses using Slope/W were performed in accordance with the Bishop’s simplified method of slices. The stability analyses of the original slope and the slopes covered with Orange Jasmine and Vetiver grass were carried out by incorporating the pore-water pressure measured during the period from 29 July 2010 to 5 January 2011 to obtain the variation in factor of safety of the slope. The variations of pore-water pressure during the rainfall for the original slope, the slope with Orange Jasmine and with Vetiver grass are shown in Figure 101a, 101b and 101c, respectively. Figure 101d shows the factor of safety variation in relation with the rainfall events. The factor of safety of the slopes covered with Orange Jasmine and Vetiver grass were relatively higher than that of the original slope during rainfall events, indicating the effectiveness of vegetative covers in minimizing rainwater infiltration into the slope. As a result, the shear strength of the slope was maintained during rainfall.
Figure 101: Pore-water pressure measurement at various depths of: (a) original slope, (b) the slope with Orange Jasmine, (c) the slope with Vetiver grass, (d) factor of safety variation, and (e) rainfall intensity from 7/29/2010 12:00 to 1/5/2011 17:50 (Rahardjo et al., 2014)
Conclusions

Thirty-one residual soil slopes from sedimentary Jurong Formation (JF), Bukit Timah Granite (BTG) and Old Alluvium (OA) have been investigated in this project. Laboratory tests (saturated and unsaturated) were carried out on soil samples for determination of the index and engineering properties of the investigated residual soils (i.e., soil-water characteristic curve, permeability function and saturated and unsaturated shear strength). The results of laboratory tests were incorporated into numerical analyses to reproduce the field variation of pore-water pressures and factor of safety for residual soil slopes from JF, BTG and OA during rainfall and dry period. The critical factor of safety for each residual soil slope was used to quantify the risk of slope failure due to rainfall and to determine the appropriate slope preventive measures for protecting the slope from rainfall-induced slope failures.

The investigated residual soil slopes were instrumented with piezometers, inclinometers and tensiometers to obtain the variations of groundwater table position, deformation and pore-water pressure within the residual soil layers. Three residual soil slopes from JF at Slope 1-3, BTG at Slope 1-7 and OA at Slope 3-7 were fully instrumented with real-time and on-line monitoring systems to provide the changes in pore-water pressure, rainfall and groundwater level continuously during rainfall and dry periods. These changes were also simulated using numerical analyses in order to quantify the response characteristics of the residual soil slopes to rainfall.

Parametric studies using numerical modelling were also carried out to study the effect of groundwater table position, rainfall characteristics (i.e., intensity, duration and pattern) and soil properties on the stability of residual soil slopes from JF, BTG and OA.

The following conclusions are drawn from laboratory results, field monitoring results and numerical analyses:

1. Soil-water characteristic curves and permeability functions are the main parameters required for seepage analysis, while the slope geometry, soil density and the saturated and unsaturated shear strength of the soil are the key factors needed for slope stability analysis.

2. The variations of soil properties with depth (i.e. index properties and engineering properties) indicated that the residual soils from BTG and OA are more uniform and coarser than those from JF. These differing properties lead to the larger variation of groundwater table between dry and
wet periods for residual soil slopes in the JF as compared to those in BTG and OA.

3. The variation in the factor of safety of a slope during rainfall was determined by the application of rainfall to the numerically modelled slopes using the laboratory and field determined soil properties.

4. The residual soil slope from JF at Slope 1-3 had a shallower initial groundwater table, resulting in a lower initial factor of safety than the residual soil slopes from BTG at Slope 1-7 and OA at Slope 3-7. A faster rate of increase in pore-water pressure due to rainwater infiltration was observed in the residual soil slopes located within the BTG and OA as compared to that in the residual soil slope located within the JF due to the higher permeability of the residual soil of the BTG and OA as compared to that of JF. Consequently, the factors of safety of the residual soil slopes of BTG and OA decreased at a faster rate during rainfall and also increased at a faster rate after rainfall stopped as compared with the factor of safety variation of the residual soil slope of JF.

5. Antecedent rainfall events with sufficient durations, frequencies and intensities could affect the stability of both high conductivity and low conductivity soil slopes, by lowering the factor of safety of the slope prior to the occurrence of subsequent rainfall events. The shape and magnitude of the different patterns of antecedent rainfall events controlled the rate of decrease in factor of safety, the minimum factor of safety and the time corresponded to the minimum factor of safety. The rate of decrease in factor of safety was faster for the advanced rainfall pattern followed by the normal and delayed rainfall patterns. For high conductivity soil slope, the delayed rainfall pattern resulted in the lowest minimum factor of safety because the amount of infiltrated rainwater was the highest among all the patterns associated with antecedent rainfall events. For low conductivity soil slope, the advanced rainfall pattern resulted in the lowest minimum factor of safety because the amount of infiltrated rainfall was the highest among all the patterns associated with the antecedent rainfall events.

6. The field monitoring results showed the effectiveness of the horizontal drains installed for slope stabilization as determined by their ability to lower the groundwater table. The numerical analyses indicated that the performance of horizontal drain in lowering groundwater table is affected
by length of horizontal drain, drain spacing, drain diameter and drain location.

7. The field monitoring results indicated that capillary barrier system was capable in reducing the rainwater infiltration into slope and in maintaining the negative pore-water pressure effectively during rainfall. As a result, the shear strength in the unsaturated zone of the slope with capillary barrier system can be maintained, resulting in less susceptibility of the slope to instability due to rainfall.

8. The field monitoring results indicated that vegetative cover (Orange Jasmine and vetiver grass) was able to maintain the negative pore-water pressure within the slope during rainfall. In other words, vegetative cover is an effective slope cover to provide additional shear strength to the residual soil and to maintain the stability of the slope during rainfall.

9. A patent (Rahardjo et al., 2012g) has been granted for “apparatus and method for preventing slope failure” that was used effectively in this project.
References


*Note: The publications written in bold font are the deliverables from these collaborative research projects.*